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Proceedings of the American Society of Civil Engineers

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USBR'S LOWER-COST CANAL LINING PROGRAM[a]

R. J. Willson,* M. ASCE
(Proc. Paper 1589)

INTRODUCTION

With the present growth of the West, which constitutes the Bureau of Reclamation's sphere of activity, water is fast becoming the most critically deficient resource of the area. In recent meetings of this Society and others, much thought has been given to water conservation nation-wide, at least, if not world wide.

In addition to the principal problem of conserving water lost in transit from unlined canals, the water lost by seepage from waterways creates another problem on some irrigation projects. Much good farm land lying below canals and laterals is water-logged by the seepage and many acres are thereby put out of cultivation.

One method of saving water and reducing seepage is to be sure that as little as possible escapes in transit to the reservoirs, irrigated land, or other point of use. The solution to this problem is to be certain conveyance systems are as watertight as it is economically possible to make them. Lining seems to best meet this criteria.

Lining waterways has been of two-fold benefit on Bureau of Reclamation projects. Water has been saved and the amount of drainage construction necessary in reclaiming good fertile crop producing acres has been reduced. Lining, of course, will not eliminate all drains, nor is lining less expensive than drain construction in many instances. It is gratifying, however, to see some of the drains on our projects being filled, and the recovered land being utilized for crops, as a result of the decrease in seepage due to canal or lateral lining.

General Policy

It has been general policy of the Bureau of Reclamation, during initial

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a. Presented at a meeting of the Irrigation and Drainage Div., ASCE, Spokane, Wash., September, 1956.

* Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

construction, to line only those canals and laterals where it was quite certain there would be excessive leakage or where the safety of the canal, its structures and the adjacent land is essential. Where there is a reasonable chance that the channel will function satisfactorily without lining, the lining has been omitted. If it later develops that lining is required, it is done as supplemental construction. Experience to date has proved this policy to be a sound one.

In the past, silt, earth blankets, and concrete have been primarily relied upon to reduce the water losses from waterways. Shotcrete also was and still is used quite extensively as a lining material, primarily in the south and southwest. Silting and loose earth blankets are not costly and have been successful in reducing losses to a point where they can be tolerated, but generally they have been only of temporary benefit. The more reliable linings in the past have generally been constructed of concrete or shotcrete, many of them being reinforced. But these materials are expensive. As a matter of fact, costs to provide permanent relief would have made the construction of some projects infeasible and the rehabilitation or protection of going projects an impossible burden to the project water users.

The Lower-Cost Canal Lining Program

In 1946, the Bureau of Reclamation, recognizing the continuing need for conserving valuable irrigation water, made a concerted effort to reduce canal lining costs. A committee was appointed and a lower-cost canal lining program was initiated. The program was and continues to be one of laboratory and field study, field installation, and field evaluation. This last is more important as time goes on, as the overall cost of a successful lining must include the cost of maintenance or replacement.

A number of materials have been studied thus far under the program and have been adapted for lining purposes; resourceful contractors have been especially helpful in assisting in the design of new equipment and procedures for adapting these materials to present-day construction methods. Manufacturers have been most helpful in the development or improvement of new materials in which their products can be used. The designers also have contributed to the development of lower-cost linings. In the case of concrete linings, savings have been effected by eliminating reinforcement steel and permitting greater tolerances in alinement, grade and surface finish.

The variety of materials studied is presented in Table I. Since the program was initiated in 1946, more than 30,000,000 square yards of lower-cost type linings have been placed in about 1,000 miles of canals and laterals on Bureau projects. One of the most important facts learned is that no one lining is suitable for all conditions encountered.

The More Commonly Used Linings

Volumewise, concrete still continues to lead the field as a canal lining material, followed in order by buried, hot applied asphalt membrane; heavy compacted earth; shotcrete; bentonite membranes; earth blankets; and asphaltic concrete. These represent about 99 per cent of the lower-cost type linings placed since 1946.

From continued evaluation over the past 10 years, it has been found that these linings, with the possible exception of loose earth, have been quite

TABLE I

LININGS PLACED

*Since December 31, 1946

	Square Yards	Miles
Unreinforced Concrete	17,406,210	389.0
Shotcrete	1,054,221	115.6
Asphalt:-		
Asphaltic Concrete	228,416	31.9
Buried Asphaltic Membranes:		
Hot-applied (sprayed)	7,000,000	330.0
Prefabricated, lightweight	15,376	1.2
Prefabricated, heavyweight	3,434	0.5
Asphalt Macadam	7,372	0.8
Prime Asphalt Membrane	13,591	1.8
Earth:-		
Thin Compacted	131,519	5.9
Heavy Compacted	4,176,235	72.0
Loose Earth	351,146	19.1
Bentonite:		
Raw or Processed Membrane	257,384	17.0
Bentonite-soil membranes	6,080	0.8
Mixed-in-place	24,664	1.8
Soil-Cement:		
Standard	69,020	8.0
Plastic	10,018	1.1
Miscellaneous:-		
Includes grouting, asphalt undersealing, asphalt-emulsion concretes, sediment linings, etc.	22,399	2.5
TOTAL	30,777,085	999.0

*Does not include reinforced concrete

efficient in reducing seepage and are the types that have given best service with a minimum of maintenance. Specifications have been developed for their construction. A continuing effort is being made to improve the specifications, the materials being used, and the methods of construction and maintenance.

Unreinforced Concrete Linings

Unreinforced concrete is generally placed in the larger canals by means of rail-guided slip-forms, similar to that shown in Figure 1. This type of slip-form is not entirely new to the placement of unreinforced concrete. Similar devices have been and are adaptable to the placement of reinforced concrete.

A subgrade-guided slip-form, similar to that shown in Figure 2 is used in the placement of unreinforced concrete linings on the smaller canals and laterals. The development of this device has made lining of small canals and laterals with concrete economically feasible.

Hot-Applied, Buried Asphaltic Membranes

Buried, asphalt membranes evolved from earlier unsuccessful surface linings of asphalt applied to the subgrade of a canal or lateral. Asphalt membranes need protection from the sun, from the weather, and from mechanical injury such as from livestock.

With reference to Figure 3, the buried asphalt membrane lining consists of a layer of asphalt, about 1/4-inch thick, sprayed at high temperature to form a waterproof barrier. This membrane is held in place and protected by a layer of earth and gravel. The hot-applied asphalt cools quickly, as shown in Figure 4, and is soon ready for application of the cover.

Heavy Compacted Earth

Third in order of volume placed is the so-called "heavy compacted-earth" lining, Figure 5. With the development of construction equipment capable of economically hauling and placing large quantities of earth, the heavy type of earth lining has been used more extensively because of its low cost for medium to large size canals. In this type of lining, Figure 6, the bottom and side slopes are built up with selected impervious soils compacted in horizontal layers 6 inches in thickness. The bottom thickness on Bureau installations is usually 2 feet, but varies with the requirements of the job. The layers on the side slopes are usually 6 to 8 feet wide and this width makes it possible to use conventional large earth-moving and compaction equipment. The actual thickness of the lining on the side slopes is about 3 feet, measured perpendicularly to the subgrade slopes.

Shotcrete Linings

Shotcrete, Figure 7, is a mortar consisting of sand, portland cement and water applied to the canal subgrade by means of an air jet. The shotcrete process is well adapted to lining a canal to any desired shape or lining thickness, and may be used on irregular subgrades. The small size and portability of the placing equipment make the method very useful for small or widely scattered jobs. The mortar layer in Bureau work is usually 1 1/2 to 2-inches in thickness.

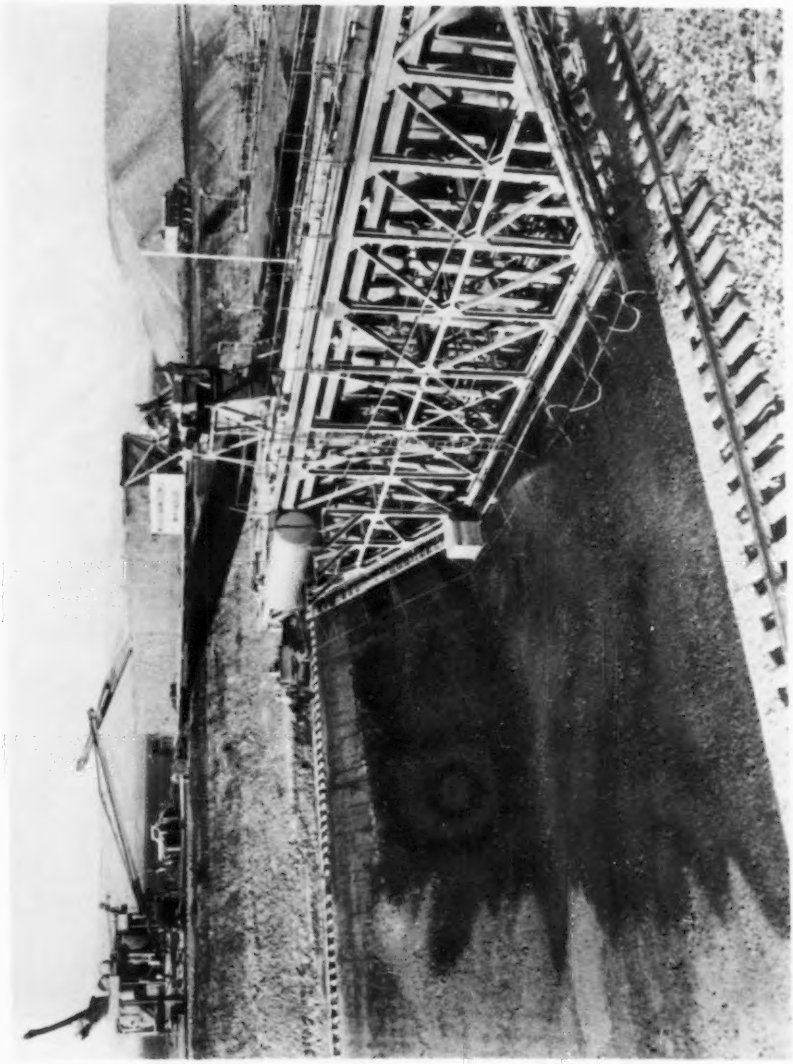


Fig. 1

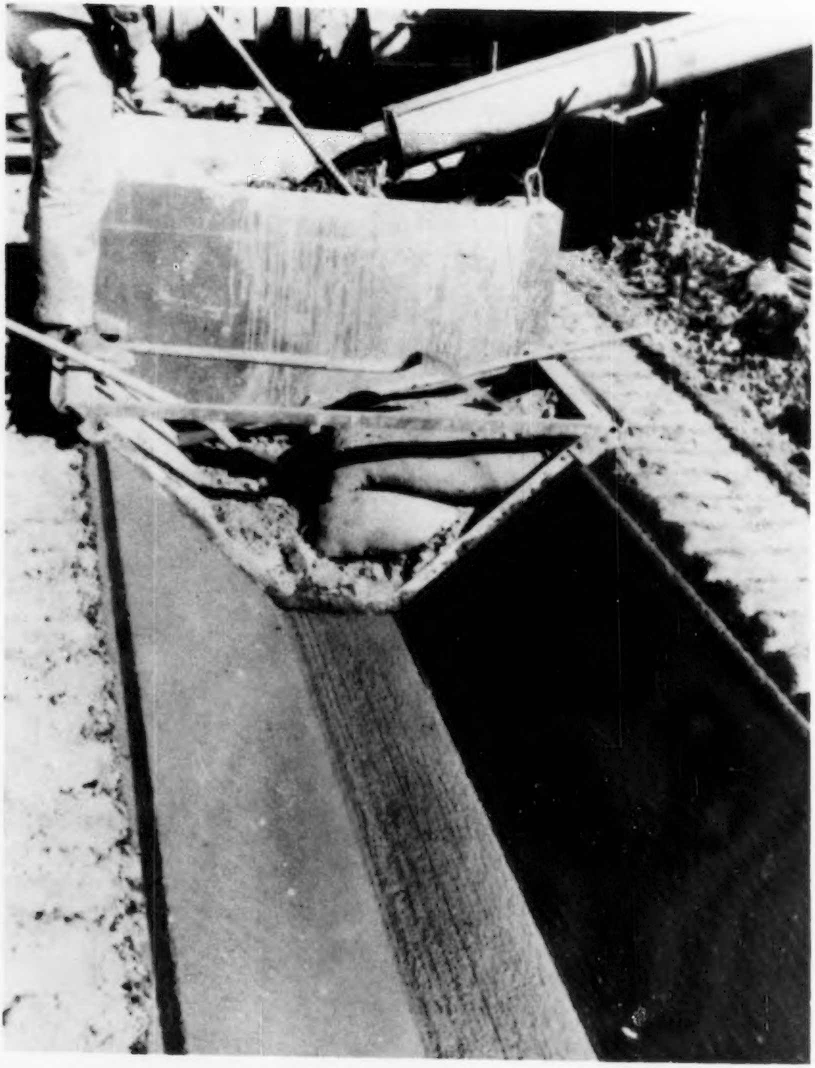


Fig. 2

ASPHALT BURIED MEMBRANE CANAL LINING

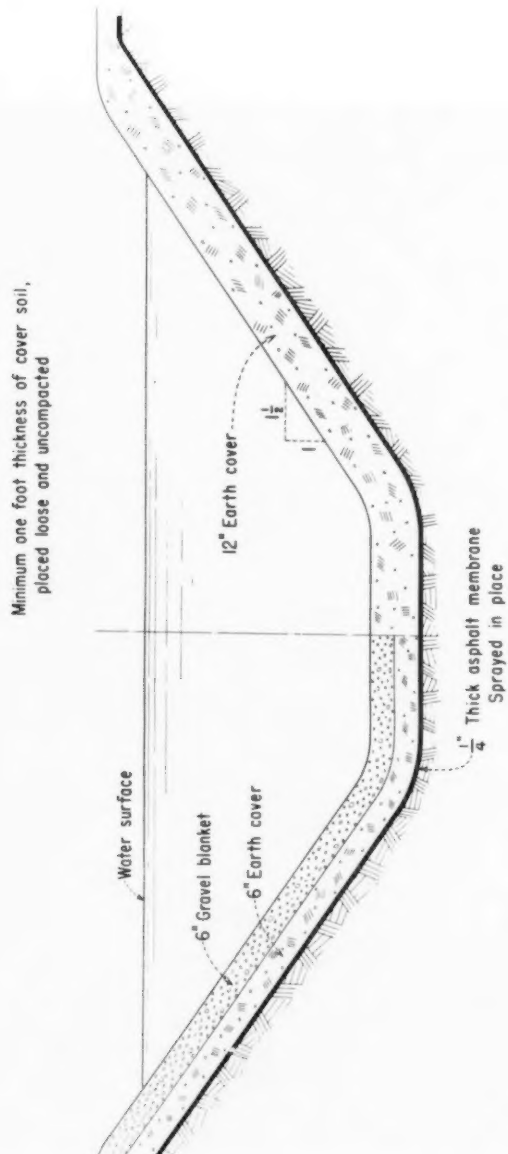
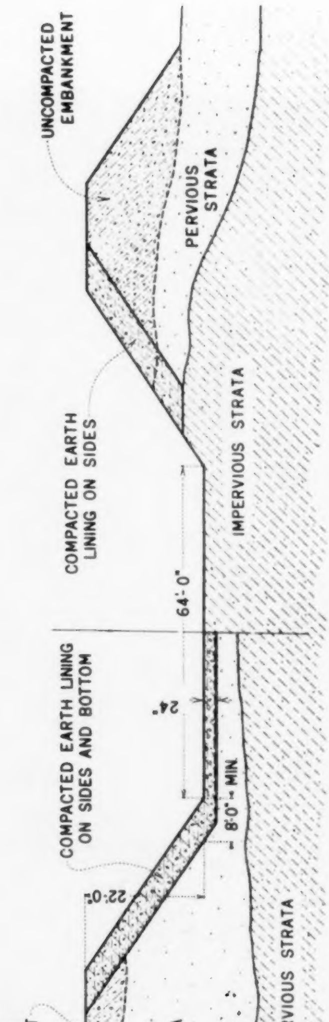


Fig. 3

Fig. 4

Fig. 5



TYPICAL CROSS-SECTION OF HEAVY COMPACTED EARTH LINING

Fig. 6

Fig. 7

Buried Bentonite Membranes

Bentonite is an earth material containing a large percentage of one of the clay minerals of the montmorillonite group, which group is characterized by the properties of swelling when wetted and imperviousness. These characteristics make bentonite a very useful material in the control of seepage from canals. The bentonite used, Figure 8, may be a pit run product screened to remove larger chunks, or it may be the commercially processed dried, granular, or powdered form. The buried bentonite membrane is similar in some respect to the buried asphalt membranes in that the 1 1/2- to 2-inch thick layer of bentonite spread over the subgrade must be protected from erosion or damage by an earth and/or gravel blanket.

Loose Earth Blankets

This type of lining is mentioned here only because it is one of the types of linings used in some volume by the Bureau because of its low cost, or its convenient use in an emergency. These very inexpensive linings consist of a loose (uncompacted) blanket of selected fine-grained soil dumped in the canal and spread over the bottom and side slopes. Unprotected by gravel the loose-earth linings are subject to erosion and severe damage from maintenance. There are few records indicating that the linings have been permanent or that they have been efficient in reducing seepage over long periods of time. It has been found in many cases that a more permanent lining is installed, as soon as the irrigation season permits or funds can be made available.

Asphaltic Concrete

The hot-mix type of asphaltic concrete, which has long been used to such good advantage for street and highway paving, was given early consideration by the Bureau to determine its possibilities as a satisfactory low-cost canal lining material. It is particularly well adapted to smaller canals and laterals, which like reinforced concrete linings in similar locations, permits the use of an inexpensive subgrade-guided slip-form.

Evaluation

The linings discussed so far are fairly well established so far as suitability for the purpose is concerned. They all have a place, no one is suitable under all conditions for all locations, and they all must be maintained. Some of the advantages and disadvantages in using these more commonly accepted types of linings are discussed in the following paragraphs.

Concrete Linings

The placement of concrete lining by subgrade-guided slip-forms developed to date becomes less practical as the cross-sectional area of the canal exceeds 50 to 60 square feet. Probably the maximum size canal economically feasible for lining in this manner is one in which the area does not exceed 100 square feet.

Although the rate of placement of concrete by a subgrade-guided slip-form will vary considerably due to many factors, the controlling factor in most instances has been the rate at which concrete can be supplied to the slip-form.

454-622-995 - Placing earth cover on bentonite membrane lining at Station 98+00. Clods on bentonite membrane surface in foreground were tossed out of Screed during spreading operation.

Fig. 8

As in the case of any concrete structure, the subgrade conditions upon which the concrete lining is placed is of considerable importance and it should be well consolidated, or if backfilling is necessary, the material used should be well graded to prevent its "piping."

Concrete linings placed over expansive clays also have created problems. The clay must be removed and replaced with sandy or gravelly material free of clay before placing the concrete lining or, preferable, another type of lining should be used. Also, an attempt should be made to avoid placement of concrete and shotcrete linings in a location where they will be subjected to a high external hydrostatic pressure or where the pressure coupled with subsequent freezing and thawing could result in heaving by the subgrade soils.

It has been found that construction joints or dummy groove control joints in concrete and shotcrete linings, spaced at 6- to 10-foot centers, aid in controlling and reducing the number of cracks and the amount of spalling that subsequently occurs.

Buried, Hot-Applied Asphaltic Membrane Linings

Side-slopes no steeper than 2:1 are favored for buried, hot-applied asphaltic membrane linings, because of erosion and beaching action. Although slopes steeper than 2:1 have been used in some of our earlier installations, it has been noted that they generally are now approaching 2:1, regardless of the angularity of the cover material. The traffic of livestock, which in open range enter the canals for water or cross and recross going to and from pasture, affect the stability of cover material on the slopes. The sloughing of the cover material down the side slopes can be controlled by using less sandy cover materials.

Although many types of asphalt have been used experimentally, it has been found that a membrane composed of a catalytically-blown asphalt has given best results to date. Asphalt in which the catalyst has been phosphorous pentoxide appears to exhibit less deterioration with age. This asphalt has a very low temperature susceptibility, a high degree of toughness or resistance to tearing or breaking, and apparently a longer life. However, aging is not considered to have contributed to failure of the membranes thus far, although some linings have been in service 8 years.

In many of the earlier installations, asphalt was applied at the rate of 1.00 gallon per square yard. To insure better coverage as well as a thicker membrane, a minimum asphalt application rate of 1.25 gallons per square yard is now specified.

Heavy Compacted Earth Linings

A properly constructed heavy compacted-earth lining has been found by actual field tests to be highly impermeable and to have water losses of about 0.07 cubic foot per square foot of wetted area per 24 hours. This loss is comparable to that of a good concrete lining. In areas where expansive soils or high ground-water are prevalent, this type of lining may actually be preferable to other linings, because of its greater weight and flexibility.

Although heavy compacted-earth linings appear more suitable for large canals, some adaptations to small canals and laterals have been successfully made. A narrower lining on the side slopes can be built with the same thin layers having a horizontal width of only 3 to 4 feet, the lining in this case being compacted by a single sheepsfoot roller pulled by a small crawler or

farm-type tractor. Another method that has been used in small canals or laterals is to fill the canal prism with compacted-earth and later re-excavate it with a plow-type ditcher, as illustrated, Figure 9.

To determine the effect of climatic changes on the durability and performance of compacted earth canal linings, density and permeability tests are performed periodically on the lining material in the field so that any possible changes in soil properties can be detected. Recent tests on a canal in Colorado, show none to a small decrease in density of the lining over tests made a year before. The decrease in density has usually occurred in the upper few inches of the lining. It is known that linings of this type on one project in Wyoming have been in service 15 to 20 years without material decrease in density.

On the Columbia Basin Project, to overcome an absence of suitable compaction material, a finer soil is blended with the sandy and gravelly subgrade material. These blended linings are proving to be very serviceable and in the low-cost field.

Shotcrete

Shotcrete linings have given satisfactory service in many installations for 20 years or more where the temperature variation is not extreme, where water can be kept in the canals year around to reduce temperature variations and the subgrade material is stable. In northern climates shotcrete linings have not been so successful. They are not considered as desirable or as economical as slip-form placed concrete linings for large jobs. The rate of placement is very slow compared to slip-form concrete operations, and shotcrete 1 1/2 to 2 inches in thickness usually costs as much as 2 inches of concrete, if conditions are favorable to slip-form placement.

In Washington and New Mexico, shotcrete linings have been damaged seriously by cracking and buckling due to expansion, where the lining is alternately exposed to the sun and to wetting and drying. Sawing joints transversely across the lining and filling these joints with asphalt has relieved the cracking and buckling by allowing for expansion. There is evidence that wire mesh reinforcing adds to the permanence and reduces the amount of maintenance repair necessary.

Shotcrete does have a place in the southwest and also is an economical method for the repair of concrete linings. Careful control of and securing a uniform lining thickness is especially important.

Buried Bentonite Membranes

Records of good service for buried bentonite membrane linings date back to 1941. The bentonite was spread to an average thickness of 1 1/2 inches over the subgrade before being covered with about 8 inches of stable earth and gravel as a protective blanket. Samples taken from the bottom of canals after having been in service show the bentonite to have swelled to a compact gelatinous mass that provides a very effective barrier to the passage of water.

It is important that the bentonite at the time of construction be spread uniformly over the subgrade and protected by cover from damage. Side-slopes on which the membrane is placed should be no steeper than 2:1, as for other buried and protected membranes. Care and maintenance of the cover are equally important.

Fig. 9

Asphaltic Concrete

The first cost of this lining is about the same as Portland cement concrete placed by slip-form methods. The linings of this type have given good service on the Bureau's Yakima and Owyhee Projects for 12 to 18 years. Some types of weeds can penetrate asphaltic concrete. In fact, some plant growth appears to be actually promoted by the heat-absorbing property of the black surface. Treatment of the subgrade with a soil sterilant prior to placement of the lining is advisable.

The asphaltic concrete lining is also characterized by uniformly spaced shrinkage cracks which continue to develop for some time. They must be repaired, preferably with hot asphalt.

A considerably harder asphalt is specified in hot-mix linings today to obtain better resistance to water erosion and weed puncture.

Experimental Linings

Several other types of linings and lining materials have been tested, as shown in Table I. These are considered to be in an experimental stage and are being evaluated. They include prefabricated asphalt membranes, asphalt macadams, prime-asphalt membranes, asphalt-emulsion concrete, asphalt-emulsion sand-mortars placed pneumatically, thin-compacted earth, bentonite-soil mixtures, soil-cement, plastics, and chemicals. Chemicals, asphalts and other materials also are being studied in soil-stability tests. Asphalt has been used to underseal concrete linings.

Prefabricated Asphaltic Membranes

Probably one of the most important developments so far as a convenient lining for placement by operation and maintenance forces on small laterals is the buried lightweight prefabricated asphaltic membrane, Figure 10. It can be installed by unskilled labor and a minimum of equipment. Several manufacturers are now producing the relatively thin 1/8-to 1/4-inch thick prefabricated asphaltic material and a prefabricated asphaltic sheet 1/2-inch or more in thickness. The latter is designed for use as an exposed membrane; however, Bureau of Reclamation experience with this thicker material is limited. Several trial installations have been made and in the southwest the material appears to be standing up well after 4 years.

Recent tests made upon samples of the thinner buried prefabricated material, obtained from field installations, indicate the thinner prefabs should probably be not less than 3/16- or 1/4-inch in thickness.

Asphalt Macadams

Field experiments indicate that it is not economically feasible to make the asphalt macadam linings watertight for extended periods, because of the large quantity of asphalt required and the continued maintenance necessary to fill cracks and voids that develop upon exposure and accompanying shrinkage. Several trial installations have been made in which the macadam has been placed over an asphalt membrane. This procedure provides a more watertight lining; however, the practicability of using the macadam for a cover of a buried membrane is offset by the less expensive use of earth cover material.

Fig. 10

Prime Asphalt Membranes

Prime-membranes which were actually the forerunner of present buried asphalt membrane linings are no longer considered a potential low-cost lining. They were constructed by first priming or penetrating the soil with a light fuel oil or distillate and then placing an asphalt membrane over the soil to stabilize it. For the most part, the prime-membrane has proved expensive, critical to weather and soil conditions, and subject to excessive injury by livestock.

Thin Compacted Earth

Thin compacted earth linings have not been used widely due to the relatively high cost of side-slope compaction in the past and the danger of removal by scour or in canal cleaning operations. There also is some question regarding the loss of density upon freezing and thawing in the northern climates. If the compacted lining is reduced to a loose state, it is probably more permeable and easily eroded. Some linings of this type that have been placed have generally given best results if provided with a gravelly cover to prevent erosion and scour.

Bentonite-soil Mixtures

Bentonite has been mixed with sandy gravelly materials by either spreading the premixed material over the canal perimeter or spreading the bentonite on the subgrade and discing or plowing it into the subgrade material. It has been effective when it is well compacted and where sufficient bentonite has been used to fill the voids of the sands and gravels. This has required from 5 to 25 per cent bentonite with the result in essence being a blended compacted earth lining. Extreme difficulty occurs in placing this type of lining on the side-slopes by mechanical means, and hand placement costs make it too expensive.

Soil-Cement

Soil-cement linings have not been too successful. They are not as weather resistant as concrete and securing adequate compaction of the material when placed on the side-slopes is a problem. In an attempt to secure better compaction and densification of a 5-inch thick soil-cement lining, an experiment was made on the Columbia Basin Project in the fall of 1956 to provide a greater density by longitudinal rolling with heavier rollers.

Plastic soil-cement linings as compared with the standard type have given best success in the southwest. These were actually lean portland cement mortar linings. Soil-cement offers possibilities for use as a canal lining material in localities where subgrade soils or those adjacent to the canal are of a sandy nature and other suitable materials are not readily available. Placement of the plastic type by slip-form reduces cost.

Asphalt-Emulsion Concrete

The cold-mixed asphalt linings are similar to the hot mixed in that well-graded aggregate and asphalt are mixed and compacted in place. The curing of linings of this type requires time and favorable weathering conditions. Some of the mixes tend to remain soft indefinitely while others contract in

curing, thereby creating cracks which must later be filled. Cold mixes tend to exhibit low erosion resistance and stability for an appreciable period of time after placing. In general, hot mixes are preferable to cold mixes and the cost per square yard for placement is about the same.

Pneumatically Applied Asphalt-Sand Linings

Like shotcrete, which is a mixture of portland cement, sand and water applied pheumatically, mixtures of asphalt and sand have also been placed pneumatically. The method is particularly advantageous in covering exceptionally rough or irregular surfaces where use of a slip-form is impractical. The slow rate of application, cost, and low-erosion resistance of the completed surfaces thus far are unfavorable to its use.

Asphalt Undersealing of Concrete Linings

The undersealing of old concrete linings with asphalt, although not strictly a lining in itself, is important as a method of rehabilitation. Hot asphalt-cement pumped under the lining at moderate pressures oftentimes travels 5 to 40 feet, filling voids, joints and water channels, re-establishing the subgrade support, and minimizing leakage without appreciable lifting of the lining.

More Recent Developments

Some of the more recent developments that appear at this time to offer promise and upon which studies are now underway are the resurfacing and repair of concrete linings; a new approach to sediment linings; cast-in-place concrete pipe, which is not a lining but is an alternative for lining; plastic linings; and the underwater application of asphalt sheets where water cannot be taken out of a canal to install a lining.

Repair of Concrete Linings

There is a need for a less costly method of repairing or resurfacing deteriorated or surface eroded concrete linings on many of our projects. Portland cement concrete linings have been successfully repaired in the bottom of canals with asphaltic concrete, Figure 11. These repairs have performed well over a period of 10 to 12 years. However, repairing the side-slopes where compaction to secure density is difficult has been a problem. An attempt is being made to secure better compaction and to develop new adhesives to help bond the asphalt repair to the old concrete surface.

Also, 1/4- to 1/2-inch thick-prefabricated asphalt sheets have been used to repair concrete surfaces. An installation of this material made last year is shown in Figure 12.

Sediment Linings

The puddling and priming of new unlined canals with silt, or sediment, a better and more inclusive term, is an accepted practice in reducing seepage losses from canal systems. A natural sealing of operating canals occurs if water in the canal carries considerable sediment. If the water carried by the canal is relatively clear, sediments have been added artificially.

Fig. 11

Fig. 12

The sealing of canals and laterals with sediment has been found to have been unsuccessful on many Bureau of Reclamation projects, moderately successful for a limited time on some, and outstandingly successful over a period of years on a relatively few. Sealing with sediment has been found to be more effective and permanent where gravel blankets had been placed over the canal perimeter to trap and hold the sediment. Although there are other factors that should be considered, the effectiveness of the sediment in reducing seepage appears to depend upon the suitability of the sediment; the velocity of the water flowing in the canal; the type and amount of sediment carried by the flowing water; and the structure of the soils in which the canal is constructed and through which the seepage occurs.

A natural layer of sediment, deposited on the wetted perimeter of a canal constructed in fine sandy soil reduced the seepage for a short reach from 5.0 cfs to 0.5 cfs. until it was scoured and eroded by clearer water flowing the canal. Apparently, little permanent benefit can be achieved from such sediment layers unless the sediment penetrates the voids in the subgrade material to sufficient depth to be protected from removal by scour, erosion or mechanical means.

Sedimenting is not costly and since it does offer possibilities under certain conditions, a cooperative study between the Bureau of Reclamation and Colorado State University, Fort Collins, Colorado, was initiated in July 1953 to investigate the possibilities of sediment sealing in more detail and determine the conditions under which the process could be expected to be successful.

After considering several types of sediment, a high-swelling type bentonite was chosen as the sedimenting agent because of its chemical and physical uniformity as compared with other types of natural clays; its high colloidal yield; its ability to expand upon becoming wet; and its availability at lower cost in the general area in which the first tests were made as compared with other types of materials.

In the laboratory and field studies which have been made to date, efforts have been directed toward securing stable suspensions of the colloids in the canal water by using dispersing agents, as the chemical nature of the water and the subgrade materials of the canal affect this stability; making field trials in small canals and laterals; determining the depth of penetration of the colloids into the voids of various types of subgrade materials both in the laboratory and in the field; and determining the permanence of the seal provided in both laboratory specimens and in the canal subgrade material.

The bentonite used has usually been a commercially processed, dried, ground and screened product. In the field trials the bentonite has usually been mixed with the canal water and a dispersing agent in a high velocity air-water jet, Figure 13. The mixed slurry of bentonite, dispersant and water then has generally been placed into a retaining pond where the non-colloidal and foreign particles have been permitted to settle. From the pond the slurry has been drawn or pumped into the water flowing in the lateral to secure about a 1 per cent bentonite concentration by volume, when mixed with the water flowing in the lateral. The mixture then has been allowed to flow down the canal, carrying the colloidal material into place.

Temporary reduction in seepage has been accomplished in field trials made by the Bureau of Reclamation in several types of subgrade materials where the sealing has been attempted. The Colorado State University in other field trials reports somewhat better and more lasting results. The

Fig. 13

temporary reduction has saved water and poses the economic question as to whether the value of water saved justifies repeated sedimenting or whether over a period of years a more permanent conventional lining would be less costly.

The amount of seepage reduction has varied with the character of the subgrade materials. So far bentonite sedimenting has been most effective where seepage from the laterals has been through root holes or other secondary macro pores in the soil structure, such as those encountered in loessial types of soil. In coarse sandy and gravelly materials and in fractured bedrock, the bentonite apparently is carried on through the voids or fissures by the seepage water without sealing them. From laboratory tests made upon typical finer grained sands and soils and from samples of such soils taken from the laterals after treatment with bentonite, it apparently has not been possible to penetrate the voids of this finer material to any appreciable depth with the colloidal bentonite. Apparently a surface layer of bentonite on the wetted perimeter of the laterals has been responsible for the temporary reduction in seepage, and upon erosion of this surface layer seepage has returned to about the same value as it was before sedimenting.

Because of the difficulties discussed, sediment lining must still be classed as experimental. However, research and development work on the problems are continuing.

Monolithically Cast-in-Place Concrete Pipe

On two Bureau projects, monolithically cast-in-place concrete pipe, similar to that shown in Figure 14, has been installed. This pipe and other similar pipe has been used by private irrigation districts in California and has a good record of serviceability. The pipe has its limitations and has been used primarily to replace small laterals and lateral distribution systems having hydrostatic heads up to about 8 feet maximum.

The cost of this and other types of concrete pipe in the past, compared to the placement of linings in open canals, has been high and has been one reason for their limited application on Bureau projects. The recent installations will provide some additional information regarding the cost and serviceability, but preliminary data indicate that cast-in-place pipe in the smaller sizes costs less than precast pipe and is actually competitive with concrete slip-form linings under favorable conditions.

The special machine used to construct this type of pipe operates in the previously excavated trench and pulls itself forward by a winch and cable anchored several hundred feet away in the bottom of the trench. Arch forms to support the top of the pipe are fed into the machine as the work progresses and a man inside of the pipe places shoring to hold the forms in place. The relatively simple machine illustrated in Figure 14 is patented.

Plastic Linings

The general problem in the use of plastics continues to be cost. However, many plastics tested so far by our laboratories have low resistance to puncture and some types disintegrate rapidly upon exposure. Thicker plastics with greater resistance to these forces have been considered and some are under test; however, the thicker material is more expensive.

The plastics do have a high resistance to rupture and rot, and some placed as buried membranes in 1955 are performing well. The future of plastic

Fig. 14

linings will depend upon results of investigations now underway and on the cost and availability of material as it is developed. The manufacturers are working with the Bureau and others in an attempt to solve the problems.

Underwater Installation of Asphalt Sheets

There are hundreds of miles of unlined canals and laterals on Bureau projects in Arizona and California that must be kept in service 12 months each year. Accordingly, the water cannot be taken out of the canals long enough to place a conventional type lining. A means of lining the canals and laterals without removing the water has been a very real problem. A new approach to the problem was attempted, Figure 15. One-half-inch thick prefabricated asphalt sheets furnished by two manufacturers were used in the trial installation.

The canal in which the installation was made was 30 feet wide and at time of installation the water flowing in the canal was from 3 1/2 to 4 1/2 feet in depth with velocities varying between 1 and 3.0 cfs. Although several methods of installation were tried, the most successful consisted of fabricating a panel of asphalt sheets as shown, Figure 16, starting from the top of the canal bank, down the slope and out onto a barge. The barge was then pulled laterally across the canal and the fabricated panel allowed to drop into the water. A similar operation from the opposite bank left a lapped center joint which was expected to seal when lapped over the mastic along the outer edge of the first panel placed.

Some difficulty was experienced in sinking the panel because of the current, and weighted rollers were used in overcoming this difficulty.

Following the installation the canal was unwatered and inspected. All transverse joints to the water line were satisfactory and effectively made, and of the several types of mastic used in making the joints below water, a hot-melt asphalt appeared best. However, difficulty was encountered in securing a good joint between the two panels placed from opposite sides of the bank. The problem was outlined to representatives of a corporation manufacturing staples. They advised that a large air-driven stapler for the building trade which would require but minor modification could be used to stitch the two panels together to assist in making the longitudinal joint in the middle of the canal and elsewhere if needed.

Development work remains to be done, particularly in finding a fully satisfactory adhesive and fastening process for bonding the sheets together. The material manufacturers also are confident that the lining of canals and laterals while in service by this method is feasible and low in cost. However, cost under the circumstances may be secondary if the demands for decreasing seepage are pressing.

SUMMARY

The Bureau's Lower-cost Canal Lining Program has been instrumental in bringing about lower costs and improved linings. Some results of this program are evident in almost every project the Bureau has built in recent years. Not only have costs been reduced but for that reason, more linings have been installed and more water and land has been saved. It may be that progress in the future will be slower than that achieved in the past 10 years. Nevertheless, there is much that needs to be done, particularly on sediment linings,

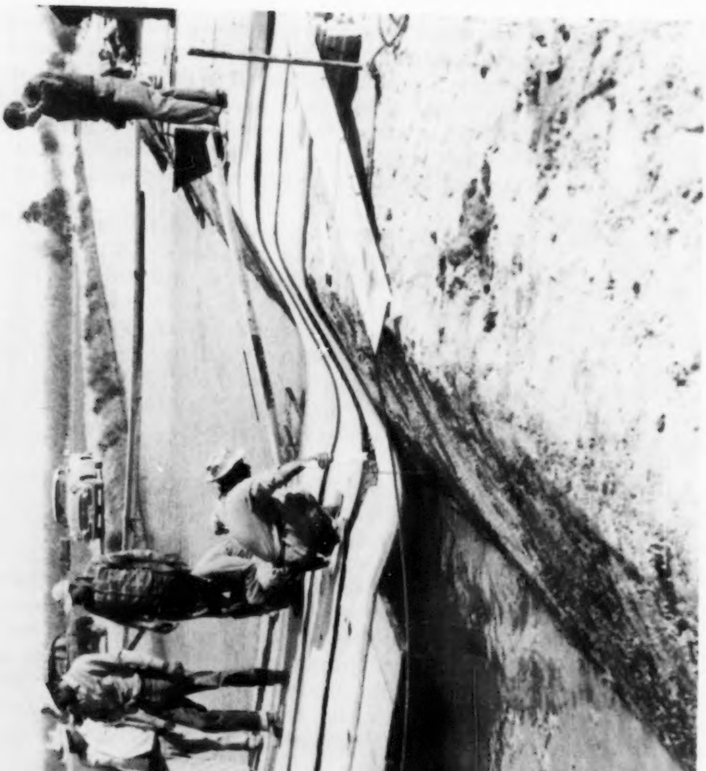


Fig. 15

Fig. 16

repairs to old concrete linings, and cast-in-place pipe. There is need also to keep abreast of new developments, particularly in plastics and chemicals. Most of the linings installed have a definite place in a program of this nature and have accomplished the purpose for which they were installed. Some have proved to be too costly for large-scale installation, have not given satisfactory performance, or, although low in first cost have required excessive maintenance. We are continuing to observe the linings installed and to evaluate them. The most successful linings will be evaluated not only on the basis of first cost, but also for the cost of maintenance and useful life.

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WATER YIELDS AS INFLUENCED BY WATERSHED MANAGEMENT[a]

Robert H. Burgy[1]
(Proc. Paper 1590)

ABSTRACT

Hydrologic studies on small brush-covered watersheds in the Coast Range and Sierra Nevada Mountains of California show that appreciable increases in runoff can result from replacement of brush by grass. Water yield increases of as much as 10 inches have been measured under favorable conditions without serious acceleration of soil erosion. These management practices also result in improved forage production on previously marginal land. Factors in precipitation disposal are discussed in relation to the influence of watershed vegetation on runoff.

INTRODUCTION

Watershed management is a term which has been used to describe many processes applied to watersheds. In its broadest sense watershed management may embrace the wide variety of management programs from the control of the area for the production of minerals, livestock, forage, timber, for recreation and certainly for water production. In the western United States as in all arid areas of the earth this latter consideration has been of principal concern. It is to the matter of the management of certain of our water-producing lands that this discussion will be devoted.

A brief look at the hydrologic cycle will reveal certain basic facts which must be considered in any study of the disposal of precipitation on a watershed. Precipitation falling on a typical watershed is disposed of in several ways: as direct runoff, indirect runoff, accretion to groundwater and soil moisture, or through loss to the atmosphere by evaporation. In each of these

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1. Asst. Prof. of Irrigation, and Asst. Irrigation Engr., Dept. of Irrigation, Univ. of California, Davis, Calif.

categories there are numerous subdivisions. If the principal interest is in the management of watersheds for the production of water and increased water yields, then the management must take a form which will modify the proportion of the precipitation channeled into each category.

It is obvious, however, that if a management program is to be of greatest benefit, it must permit increases in water yield without deleterious effects. Perhaps the most practical method of modifying disposal of precipitation on the watershed is through the management of the vegetation. This does not, however, preclude certain programs which have been used in some areas to produce maximum water yields through complete paving of the soil surface.

Vegetative Manipulation

Manipulation of vegetation as a form of watershed management has long been of interest in the field of hydrology. Numerous experiments have been conducted to determine the responses of watersheds subjected to various treatments. In the opinion of the writer it seems quite clear at this time that the manipulation of vegetation will result in marked changes in precipitation disposal factors. Removal of the primary intercepting vegetative canopy permits a greater amount of precipitation to reach the soil surface. This item alone must result in a greater contribution of water from the area since for a given set of conditions of soils, geology, vegetation and climate the balance will have been disturbed. Furthermore, the removal of vegetation in many cases may result in a modification of the soil surface, the governing agent in infiltration. Within the limits of practicality it is impossible for man to affect a modification of the actual soil capacity for retaining moisture. Thus, increased runoff must be the anticipated result of manipulation of the vegetation on watersheds where precipitation is sufficient to satisfy soil moisture capacities. The actual method of vegetative manipulation is also important in determining the ultimate response.

A further effect of vegetative manipulation is that of reduced evapotranspirational losses after converting from dense, deep-rooted brushy species to a grassy type of cover. Many grasses are less deeply rooted and may persist less vigorously throughout the summer season. Thus on the deeper soils and on watersheds where the soil moisture deficit is satisfied annually there may be a carry over of moisture with a consequent savings of water.

As was noted earlier, management must produce a beneficial response in increased water yield without undesirable effects. It would be wonderful indeed to be able to simply modify the vegetation on a watershed and obtain increases in water yields. Unfortunately, this is not generally the case. The increased runoff occasioned by the additional precipitation arriving at the soil surface and contributing to a greater quantity of surface and subsurface runoff may create situations of increased movement of debris down channels. If infiltration rates have been modified, further increases in surface runoff may cause higher erosion rates.

Studies by the University of California

Recognizing these principles, the University have been engaged in studies for some time in an attempt to develop vegetative manipulation techniques that avoid the undesirable aspects and to take advantage of those desirable ones. Within the state of California there are several major zones of vegetation on the mountain watersheds. These are generally described as the woodland

grass areas at the lowest elevation in the foothills: the chaparrel or brushlands lying in the intermediate elevations; and the forested lands at the upper reaches of the watersheds. It is within the second zone that a considerable amount of research has been centered.

Some 13,000,000 acres in this state are so situated with regard to climate and topography that they might well be considered as potential range lands to be used in the production of forage for livestock. Broad programs have been developed in attempts to control the undesirable chaparral growth on these areas and to substitute desirable forage species. Such procedures have been referred to as range management or improvement programs. These potential range lands which are being subjected to management are extremely critical due to their location, lying as they do between the areas of use in the irrigated valleys, and the upper watershed.

The Department of Irrigation of the University of California began hydrologic studies of the effects of brushland conversions in about 1933. The earlier phases of the operation involved the use of small plots situated in typical brushlands of California. These plots were established in pairs one of which was immediately subjected to a complete vegetative removal process. The other of the pair was held as a control or check plot. Some 40 pairs of these were established. Records of precipitation, runoff and erosion were collected continuously for approximately 10 years on each of the plots. The pairs were then reversed. The original treated plot was allowed to return to native vegetation and the brush-covered plot was converted to grass. In these early studies no attempt was made to revegetate the plots after removal of the brushy vegetation. Native grasses usually became established.

As the study progressed it became obvious that plots would not give a complete picture of the situation since they did not involve an entire unit in nature. Therefore, a series of small watersheds were established in the same general regions as the plots. These small watersheds consisted of pairs again. Because of the difficulty of locating two identical units in nature it was necessary to calibrate these watersheds to determine their relative hydrologic characteristics. Calibration was established at approximately 5 years on the presumption that within the 5 year period a reasonable experience of weather phenomena might be expected. As it developed, however, longer calibrations were used in the earlier studies. On both the plots and the small watersheds soil moisture samples were taken at frequent intervals throughout the season. At the end of the calibration phase of this study, one of the pair of watersheds was selected for treatment. These treatments generally consisted of the removal of the native brushy vegetation and the substitution of an improved grass-type cover.

Further experience again indicated that larger watershed units were necessary. There are several factors which enter into the selection of the most desirable size of watershed to study in such a program. One major concern is the cost of developing the equipment necessary to make the measurements. In this study erosion measurements were made by actual sedimentation of the debris coming down the channel of the stream. This debris is weighed out after each storm. The stream gaging stations used in conjunction with these sedimentation basins incorporated standard hydraulic measuring devices to avoid calibration difficulties and to permit as high a level of accuracy of measurement as was possible. In some cases volumetric measuring devices as well as weirs or flumes were used.

The largest complete watershed unit which has been developed thus far is approximately 200 acres in area. The sedimentation basin required for this watershed has a capacity of about 1/3 acre foot for debris storage. Larger watersheds have been included but these do not have debris measuring devices.

It might be noted here that the use of other sediment measuring devices such as aliquot samplers may be more economical and possibly more desirable since the labor required to service a sedimentation basin becomes appreciable. Devices of this type are being considered at the present time and may be incorporated in some of the existing structures to test the various types.

The vegetation management processes applied to these watersheds were designed to insure a complete treatment and to maximize the severity of the treatment. Certain preparatory steps were used. These involved the slashing of the vegetation early in the summer season, the slashed brush being allowed to dry on the ground for as long as possible before the application of fire to the area. Fire was used on all of the areas except one. Use of fire is indicated on this type of land cover as being the most economical means of removal of the vegetation. This preparation of the vegetation results in a much more severe treatment than would be possible otherwise. After the treatment had been applied a transitional period follows. A three-year minimum period is usually necessary to allow for a complete conversion from the brushy species to a new cover. The revegetation process is accomplished by reseeding the areas with grass species indicated to be suitable in these regions from agronomic studies.

Results of Experiments

At the present time seven major watershed installations involving either single watershed complexes or pairs are under study. These are located from in both the foothills of the Coast Ranges and Sierra Nevada Mountains. Five of these areas have been subjected to vegetation management treatments. On three of the areas the treatment has been in effect for a period of over three years. On the fourth the vegetation was removed in the fall of 1956 so only one season's record has now been gathered, and on the fifth the vegetation was removed over a period of about 1-1/2 seasons and involved a somewhat different type of vegetation management, the stripping of vegetation by means of mechanical equipment. As noted earlier, it is necessary to have several years of record after treatment to permit rational analysis. Therefore, only those which have been operated through several seasons will be considered here.

The use of mass-curves of runoff have been chosen to present the relationships between pairs of very small watersheds, one of which has been treated. Figs. 1 through 4 show the relationship between cumulative runoff in inches for the several seasons of operation. The cumulative precipitation curve has also been shown on the graphs to indicate the variation in seasonal precipitation. In all of these cases the time of the treatment is indicated on the graph.

The longest period of record is represented by the Ono watersheds. From this record we may observe that for the treated watersheds, A and D, the curves lie below the adjacent B and C curves prior to the accomplishment of the treatment. However, after treatment a reversal takes place and the D watershed particularly will be observed to show a marked increase in total seasonal runoff. Watershed A also indicates an increase in runoff by coming into position two in descending order at the end of the record. It will be noted

from the graph that runoff increased in all cases on the mass curves but that for the two treated watersheds the increases are significantly higher during the post-treatment period, even for those years of extremely low rainfall. This indicates that the vegetation management process has caused an increase in water yield or runoff even during the lowest of rainfall years.

The Tulare record, Fig. 2, shows relatively comparable runoff data up to the 1952-53 season. Thereafter the yield from the A watershed begins to increase more rapidly. The actual treatment on Tulare watershed A was accomplished during the 1955-56 season and yet you will note a slight increase in the earlier record. Perhaps this may be explained by the fact that wide firebreaks were cut around these watersheds. Additionally the A watershed lies at a slightly higher elevation and receives a greater amount of precipitation in some seasons than does B.

Fig. 3 for the Ahwahnee Stations in Madera County represents another condition wherein the runoff during the season immediately following the vegetative conversion was sharply increased on A as opposed to zero runoff on B. Precipitation during the 1953-54 season was normal for this area. During 1954-55, however, precipitation was somewhat below normal which resulted in zero runoff from both watersheds. In the succeeding 1955-56 season which was notable in California as a high rainfall year the runoff from both A and B was appreciable but with the greater quantity coming off of the B watershed. However, because of the previous runoff the total record is higher for the A watershed. This greater runoff from the brush covered watershed B leads to the conclusion that during storms of excessive duration and precipitation rate, 14.3 inches in 4 days, where saturation of the watershed mantle is attained, vegetation management effects are completely obscured by the excesses of runoff which occur. This effect has been observed at other locations.

It might be of interest to note that in this above-mentioned storm, watershed B produced erosion at the rate of 1400 pounds per acre while that from the grass covered A watershed reached only 985 pounds per acre.

Fig. 4 is that of Diamond Range located in Tehama County. At Diamond Range the record begins with the 1942-43 season and progresses on upward with the B watershed curve well below that of the A record up to the time of treatment. This treatment was accomplished in the 1953-54 season and it will be noted that the B record immediately responded with a slight increase in runoff. This increase was not noted in the 1954-55 season which was slightly below normal precipitation. However, in the 1955-56 season again a large increase in runoff is indicated on the graph.

From an interpretation of these graphs definite runoff responses from vegetation management on these typical brush covered watersheds are indicated. The whys of this response in increased water yield have been discussed earlier and are undoubtedly related to the removal of the intercepting canopy. Certainly the soil, the vegetative cover, geographical location of the study area, and the character of the storms producing the runoff all have a profound influence upon the hydrologic responses of these watersheds. During those seasons immediately following treatments when normal or greater than normal precipitation was experienced increases in runoff were noted from the treated watersheds. On all of the watersheds the areas are very small. These data represent principally surface runoff. This is important since none of the subsurface contributions from the watersheds could be measured.

It appears evident from these considerations that vegetation management may have an effect upon the runoff characters of watersheds so treated. The

data available at the present time are only for small watersheds ranging in size from less than one to approximately twenty acres. Current studies include watersheds up to 4,000 acres in area which will, it is believed, give a more realistic picture of the situation. As was suggested earlier the scope of vegetation manipulation for watershed management is rapidly developing. Further studies along these lines are now under way and will be continued in an effort to isolate in greater detail the effects of various management practices on the hydrologic characteristics of watersheds.

Figure 1. Accumulated Seasonal Runoff in Inches for Ono Watersheds

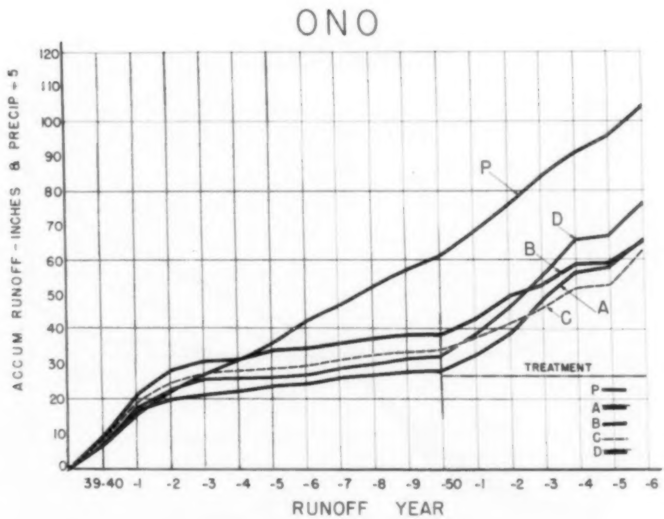


Figure 2. Accumulated Seasonal Runoff in Inches for Tulare Watersheds

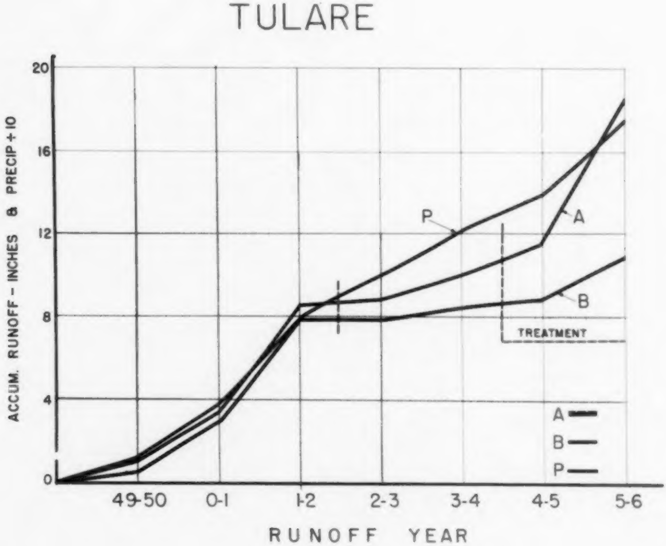


Figure 3. Accumulated Seasonal Runoff in Inches for Ahwahnee Watersheds

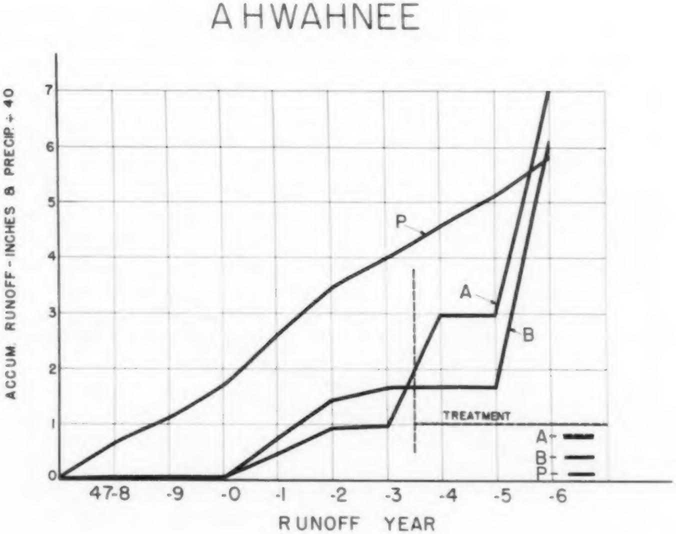
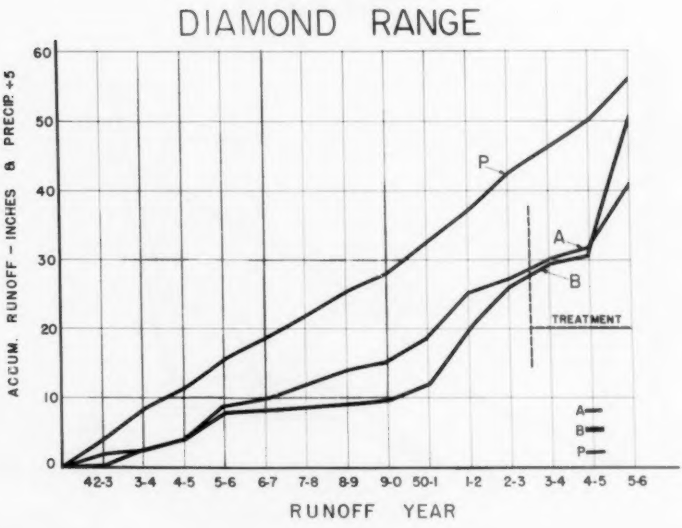


Figure 4. Accumulated Seasonal Runoff in Inches for Diamond Range Watersheds



Journal of the
IRRIGATION AND DRAINAGE DIVISION
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IRRIGATION IN NEW JERSEY[a]

Robert L. Hardman,* A.M. ASCE
(Proc. Paper 1591)

The State and the Market

The practice of irrigation in New Jersey has only in very recent years reached a stage of development which has led to its recognition by water supply engineers and by agricultural experts as a very significant feature in the overall economy of the State. It should be recognized, of course, that irrigation in New Jersey is not comparable with irrigation in any of the states west of the 100th meridian, since New Jersey is a humid state, enjoying a normal rainfall of more than 45 inches. Nevertheless many a New Jersey farmer has lost part or all of a crop, and suffered serious financial reverses through lack of irrigation facilities, for reasons which will be developed.

The State of New Jersey is unfamiliar to many westerners, known only as a space which must be crossed to get to or to leave New York City. A glance at the map of the United States will reveal that New Jersey resembles a peanut in both shape and relative size. Only the states of Connecticut, Delaware, and Rhode Island are smaller in area than New Jersey, but paradoxically only New York, California, Pennsylvania, Illinois, Ohio, Texas and Michigan have a larger population. Reference to New Jersey as the "Garden State" is no accident. A straight line connecting New York City and Philadelphia goes through the neck of the peanut and divides the state geographically and geologically. The northern half, west of the highly developed metropolitan area, is a playground for residents of the densely populated areas of both New York and New Jersey, and includes the highest point in the state, which is only 1800 feet above sea level. This area contains dozens of recreational lakes, many of which are privately or community owned. It is well wooded and provides great attraction and opportunity for outdoor recreational activities of all kinds. The open areas in this portion of the

Note: Discussion open until September 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1591 is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. IR 2, April, 1958.

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*Supervising Engr., Div. of Water Policy and Supply, Dept. of Conservation and Economic Development, State of New Jersey.

state are predominantly used for field crops, pasture, and dairying, though there are many truck farms in particularly productive areas. The southern portion of the state is a coastal plain, the elevation of which is seldom more than 150 feet above sea level. This area supports a vast truck garden which supplies fresh produce to the markets of New York and Philadelphia and to the large processing industries which produce both canned and frozen foods for consumption across the nation. The ocean front from Raritan Bay to Cape May is one long continuous beach playground, and roughly in the center of the area lie the so-called "pine barrens" which are soils of low productivity covered with scrub oak and pine and of which large acreages are preserved as natural areas through state ownership as state forests and multiple-use water reserves.

The population of the state itself, together with that of its neighboring metropolitan areas, totals something on the order of 15,000,000, which is perhaps indicative of the vast local market available for consumption of New Jersey farm products. The soils of the state are of profuse variation, ranging from the sedimentary and predominantly sandy loam deposits in and through the coastal plain to red sandstones, limestones, clays and glacial deposits in the northern half of the State, which is roughly bisected laterally by the southern terminus of the glacial moraines.

Farm Products

Of the agricultural products of the state, third in importance, after eggs and milk, is vegetable crops. In dollar value perhaps the most important of these are asparagus, snap beans, lettuce, onions, green peppers, sweet potatoes, white potatoes, strawberries, tomatoes and sweet corn. Other important crops are cranberries, blueberries, peaches and flowers. It might be interesting to note in passing that 25% of the entire orchid crop of the nation is grown in New Jersey.

Water Resources

New Jersey is most fortunately situated with regard to weather. The normal annual rainfall as now computed is 45.89 inches. Distribution of rainfall across the state is fairly uniform, with no well defined patterns of excess or deficiency in any specific area. Distribution throughout the year is also sufficiently uniform that there are no well defined periods of excess or deficiency. The rainfall in any given month is just as apt to be high as low in a succession of years. In the driest year of record, 1930, there were deficiencies of as much as 15 inches in the southwestern and northwestern portions of the state, but practically none in the north central portion.

Water is available for irrigation purposes throughout the state, although in the northern portion ground water yields are apt to be small. Surface streams also are relatively small and flashy in this area but watersheds are capable of development by the creation of storage for regulation to meet the high demand rates required for supplemental irrigation during the seldom exceeded three or four consecutive weeks of rainfall deficiency. In the southern portion of the state stream flows are better sustained and ground water is abundant. Large capacity wells may have to go down as much as 400 feet in some of the farming areas, but in many others adequate supplies may

be produced at 50 feet. Fortunately the maximum growth in irrigation demand will probably occur in this area.

Irrigation Practices

At this point it may very well be asked why there should be any concern whatever with the need for irrigation in New Jersey. In answer it could be pointed out that the total value of truck crops for both fresh market and commercial processing in 1953 was $50,605,000.* In 1954, which from the farmer's viewpoint was considered a dry year, total value was $45,798,000.* In 1955, which again from the farmer's viewpoint was a still drier year total value dropped to $38,156,000.* There are of course, many factors which entered into the fall-off in crop values in these two years. But there can be no doubt that a significant contribution to that reduction was the lack of irrigation facilities. The average rainfall for the state in 1955 was 42.17 inches, which was only 3.62 inches below normal. However, the fact that most of this deficiency occurred during the growing season brings into sharp relief the plight of the farmer who has failed to provide himself with irrigation facilities.

Truck farming in New Jersey is highly competitive practice. With the advent of inexpensive and easily handled pipe after the end of World War II the New Jersey farmer, with his fellows throughout the country, found that it was economical to install irrigation equipment to supply water to his crops at critical periods in their growth, which enabled him to insure early maturity and increased yields, and in so doing to demand higher prices for his produce at the market. This eventually forced his neighbor to adopt the same practice in order to compete successfully, and might have been a comparatively slow process except for the fact that rainfall deficiencies in the growing season have occurred so frequently in the past several years, beginning with 1949. The figures in the following table were obtained from the 1954 Agricultural Census for the Middle Atlantic States, showing an increase of 210% in irrigated lands for the five-year period.

The number of farms using irrigation in these years was 1,003 in 1949, 1,775 in 1954 and 1,850 in 1955. It might also be noted that about 25% of the total, or 400 farms covering 17,500 acres, is devoted to the production of potatoes. The U.S. Census of Agriculture in 1950 estimated that the total farm land in New Jersey amounted to 1,725,000 acres, representing almost 36% of the land area of the state. It is probable that this figure has been reduced somewhat as of 1956 because of the continuing conversion of farm lands to residential developments, but it is apparent that less than 4% is now under irrigation.

Irrigation in New Jersey is done almost entirely by the overhead rotary sprinkler method. There are a few fixed type overhead perforated pipe systems still in existence on relatively small plots, and some notably unsuccessful attempts at field flooding have been made in some areas. The high cost of land preparation, even where soils are suitable seems to preclude any economic justification for this type of irrigation.

A summary prepared by the Pacific Gas and Electric Company on the cost of water application on 24 farms in California indicates that the annual cost

*Crop Reporting Service, B.A.E., U.S.D.A.

	Acres Irrigated		
County	1949	1954	1955
Atlantic	2987	5952	Not Available
Bergen	1537	1205 (loss)	"
Burlington	8387	13,779	"
Camden	863	2217	"
Cumberland	4418	9490	"
Gloucester	1690	4437	"
Mercer	1478	3249	"
Middlesex	488	4423	"
Monmouth	2140	4526	"
Morris	616	2169	"
Ocean	933	1402	"
Salem	147	2639	"
Warren	541	1180	"
(Counties with less than 1,000 acres under irrigation not listed)			
Total (21 counties)	28,117	58,912	62,500

It may be noted that in the northern portion of the state only Bergen and Morris Counties have more than 1000 acres under irrigation, and that only Bergen has shown a decrease, due probably to conversion of farm lands to residential developments.

per acre is about 20% more for surface irrigation than for sprinkler irrigation.* It would appear that the comparative infrequency of application in humid areas would produce an even wider divergence in cost. For the same reason the large mechanically propelled "walking sprinkler" lines are not economical, and it may be said that practically all of the present systems use portable aluminum pipe lines, shifted across a field by hand labor. Rotating sprinkler capacities may run from as low as 3 gallons per minute to as much as 400 gallons per minute, with spacings ranging from 40 feet by 40 feet to 200 feet by 200 feet. Current practice is to irrigate on a cycle of approximately one week, though for particular crops and soil this may vary from five to eight days. The unproven rule-of-thumb for vegetable crops is to apply approximately one inch of water per week at a rate of something less than 1/2-inch per hour. One irrigation equipment supplier estimates that capital outlay for an irrigation system may run from $100 per acre up, and that operating cost will run approximately $25 per acre for the first inch of water applied in any given year and $5 per inch thereafter.

*Ref: Sprinkler Irrigation Association - 1955.

One of the largest farming operations in New Jersey, which plans to have over 6,000 acres of vegetable crops under irrigation in 1957, and which has engaged in supplemental irrigation for many years, has a considerable variety of equipment, including permanent buried mains, galvanized steel portable mains, and portable aluminum laterals, and estimates that the cost of these systems ranges from $90 to $150 per acre and that labor costs run from about $2.50 to $3.00 per acre per irrigation.

There are no adequate data available at the present time on the actual quantities of water used for supplemental irrigation, but it has been estimated that approximately 60% of such use is from surface sources, including streams and natural and artificial dug ponds. Professor John W. Carncross, of the College of Agriculture of Rutgers University, has made an extensive farm to farm survey of actual practices. The results of this survey, when published, will provide a valuable body of factual information which has not hitherto been assembled. Although there is not now and is not likely to be in the foreseeable future any deficiency in volume of water supply for irrigation, the lack of water in many of the smaller surface streams at the seasons of the year when the need is most urgent is creating an increasingly difficult situation which has driven many farmers to the drilling of deep wells to insure a dependable supply, and has forced others in the tidal areas to utilize water which may be brackish to the point of actual damage to the crop. A glance at the map will show that more than half of the boundary of the state is salt water, almost surrounding the low lying coastal plain, which makes the problem of brackish water during periods of low stream flow one of no mean proportions.

Irrigation Problems

At a recent meeting of the Vegetable Growers Association of New Jersey, a South Jersey farmer remarked that in spite of continuous increases in wages and in the cost of almost everything in the way of consumer goods, he was still getting approximately the same price per bushel for his crop that he got 25 years ago, asking how this could be accounted for. There can be only one answer of course, which is that production costs have declined in the same period. This requirement for continuously reducing costs is, primarily, the basis for the practice of irrigation in New Jersey, and the same will doubtless be true in all of the humid states. The farmer who is now without irrigation facilities is at the mercy of the elements and sooner or later will be forced to adopt the practice much in the same way that the supermarkets and the theatres have been forced to install air conditioning to meet competition for the consumer dollar, since both are a form of weather insurance. Those who have irrigation facilities have a roughly equal quality potential, sell to the same markets and realize a profit which bears a direct relationship to their abilities to cut costs. Cost-cutting requires refined techniques. In the west where rainfall is very limited, there is presumably little concern with excess moisture. In the humid east it is almost important to avoid an excess of water in the soil as it is to prevent a deficiency. Consideration must be always given to the occasional shower that may occur even during an apparent period of drought.

Because of the necessity for refinement in technique in order to achieve the maximum potential at a minimum cost in the use of supplemental

irrigation, many studies are under way, eventually to provide the answers which will lead to a shift from the presently more or less haphazard practices to a logical water management plan on an individual farm basis. One of the agencies engaged in these studies is the Rutgers University College of Agriculture and Experiment Station at New Brunswick, New Jersey, which has set up a program in six parts: 1 - to determine the water needs of important crops under different environmental conditions; 2 - to measure the response of different types of crops to supplemental irrigation at certain stages of growth under various soil moisture deficiency conditions; 3 - to investigate all possible means of increasing the efficiency of water used by plants; 4 - to develop a practical method of determining when to irrigate certain crops; 5 - to study the effects of irrigation on the physical properties of the soil, and of the latter on the former; and 6 - to investigate the tolerances of different crops to soluble salts and other substances found in water which might be used for irrigation, and the long and short term effect of these substances on the soil.

These studies have only recently gotten under way, and no conclusive results have yet been determined. However, some very interesting observations have been made. Doctor E. R. Purvis, in cooperation with Mr. Gerow D. Brill of the U. S. Department of Agriculture Research Service has made studies on two varieties of lima beans, using dilute sea water at 1/8, 1/4 and 1/2 strength, which indicate that the addition of 12-1/2% salt water to tap water resulted in a slight increase in yield over that obtained using pure tap water. The use of 25% salt water caused some foliar injury but did not decrease yield significantly, indicating that yields increase with the chlorine content of the leaves until a level of about 1-1/2% is reached.

Another study of the effect of 3 applications of 1-inch of dilute sea water showed that at 3500 parts per million of chlorides (12-1/2% salt water) there was moderate to slight damage to beans and sweet corn, but none at all to tomatoes and radishes. At 7,000 parts per million the tomatoes suffered, but radishes continued to be unaffected up to 14,000 parts per million, or 50% salt water, even when the dilute sea water was applied to the plant foliage. A study of the yield on yellow wax beans showed an actual increase of 20% in yield with the use of salt water at 3500 parts per million over that produced by fresh water irrigation, after using four 1-inch applications at weekly intervals. This study also showed that although the salt content of the soil increased after the application of salt water irrigation, the salt accumulations from a total of 3 inches of 1/2-strength sea water were completely leached from a Sassafras sandy loam soil during the winter and spring, indicating that it may be possible to use water of considerably higher salt content than was previously thought safe, provided the number of irrigations is limited.

Doctor Nathan Willits, also in cooperation with Mr. Brill of the U.S.D.A. Agricultural Research Program, is conducting a study of the effects of irrigation at various levels of soil moisture tension for various crops and types of soils. The development of practical and sufficiently accurate devices for the measurement of soil moisture tension is essential to the use of this type of information, and it is to be hoped that such development will keep pace with theoretical findings. Certain studies have been made on potatoes in a Sassafras loam soil at three levels of soil moisture tension, 0.8 atmosphere approximating 33% moisture depletion in this particular soil, 1.2 atmospheres approximating 50% depletion, and 2.5 atmospheres approximating 67% depletion. A summary of three year average yields for this crop indicates

that there was no significant difference between the yields obtained when the root zones were restored to field capacity at any of the three tension levels, but that inadequate supplemental irrigation did reduce yields at the two higher tension levels. While still inconclusive, of course, there is some indication that the use of more water at less frequent intervals might reduce labor costs in moving pipe, by deferring irrigation, in this instance, until soil moisture tension reached 2.5 atmospheres, thereby also increasing the possibility of restoration of soil moisture by normal rainfall.

A comparatively long term study, begun in 1945, on potatoes in the same type of soil, based on a study of continuous potatoes, two year rotation of potatoes and wheat, and three year rotation of potatoes, wheat and hay, with irrigation at 1/3 inch per hour when soil moisture tension at the 8-inch level reached approximately one atmosphere, indicates that plots under irrigation in both normal and dry years showed higher yields under 2 year rotation, and that non-irrigated plots showed higher yields with 3 year rotation. Yields were significantly higher, about 15%, during the drought years, indicating the possibility that this may be due to the fact that during years of near normal rainfall the soil physical condition was adversely affected by irrigation. Aggregability was lower, soil density higher, and the soil was more difficult to work. This effect was less noticeable in the dry years.

The apparent lesser yields with irrigation during years of near normal rainfall indicate that this may be due to mismanagement and excessive application of supplemental water, showing the necessity for determining the physical properties of the various soils through an inventory of soil profiles, correlating this with practical methods of water balance determination in order to provide a means for the individual farm irrigator to determine when and how much to irrigate. This would require that the field capacity of each soil be determined to the effective root depth of the crop planted. Based on this field capacity a determination of maximum allowable depletion must be made, bearing direct relation to the stages of growth of that particular crop at which supplemental irrigation may be most effectively used. The proper quantities to be added can then be decided upon, after providing for a certain reserve in the field capacity for rainfall which may occur immediately after irrigation, in order to avoid the adverse effect on soil physical structure due to excessive application of supplemental water. The determination of this reserve storage capacity should also include the storage capacity of the crop foliage at various stages of growth and that available in the non-capillary pore spaces of the soils, which are normally drained. Consideration must be given to the physical structure of the soil in the determination of rates of irrigation. Satisfactory methods for measuring rates of infiltration have not yet become available and it is still difficult to determine accurately the optimum rate for a particular soil. Present practice usually adds too much water.

Mr. Vaughn Havens, Assistant Professor of Meteorology at Rutgers has proposed a method for timing irrigation by using climatic data to estimate water available in the soil. This involves the maintenance of a continuous water balance sheet, based on published records of rainfall and temperature, in which the potential evapo-transpiration, computed by a formula originally developed by Dr. C. W. Thornthwaite of Centerton, N. J., may be adjusted downward to account for the gradual increase in soil moisture tension which accompanies the depletion of soil moisture during dry spells. This appears to offer considerable promise, but is of course limited by the relative

scattering of adequate weather stations, and is also directly dependent upon individual soil structure information and crop rooting depths.

The Engineer and the Law

The function of the engineer in the development of irrigation in New Jersey, apart from the basic hydraulic aspects of water supply and distribution, appears to be that of evaluation of the present and future demands for water for irrigation in terms of the seasonal effects of such demands as well as in gross volume. The continuing increase in irrigation has brought with it awareness of an imminent problem in the determination of the legal status of the use of surface waters for irrigation. This is a problem which has not been squarely faced in the humid states east of the 100th meridian until very recent years, since the use of water for this purpose has been so minor as to be relatively insignificant. It has, however, reached a very significant level in New Jersey. In spite of its small size New Jersey in 1954 had more farm land under irrigation than all of New England combined. Only three other states east of the Mississippi had more, Florida, Mississippi and New York. Although New Jersey has adequate water supply in the form of normal rainfall, it is nevertheless a fact that the state does have a problem in the distribution of water, particularly in a chronological way, since rainfall is so well distributed throughout the year that stream flows are lowest during the growing season. The total volume of water required for irrigation is relatively small, but the high rate of demand during the critical periods, when superimposed on demands by public supply, industrial, and recreational interests at the very times when natural stream flows are lowest, aggravates a problem which can only be solved by the creation of adequate storage. This in turn is complicated by the fact that the state is small and congested, and that reservoir sites are at a premium due to high demands for other land uses.

The riparian doctrine of the common law does not offer to the irrigator any protection in his right to use water for that purpose, which is essentially a consumptive use. The definition of riparian lands under the common law would doubtless limit very drastically the extent of farm lands which might be otherwise accessible for surface water irrigation. The requirement under the common law for return to the stream of substantially all of the water used, under the riparian doctrine by definition precludes the use of surface water for irrigation. A farmer who has made a large investment in irrigation equipment, dependent upon a surface stream for a source of water supply, has no protection under the common law from a similar divertor upstream who may subsequently use all of the water in the stream, nor he in turn from his upstream neighbor, since none have any rights under the riparian doctrine. Numerous cases have already arisen in New Jersey where farmers, formerly dependent upon surface streams which have been dried up by upstream neighbors during the irrigation season, have been forced to the considerable expense of drilling wells to insure a dependable supply. This problem becomes acute of course where ground water supplies are not readily available. The rapid increase in the number of farms installing irrigation facilities indicates that the time is rapidly approaching when the courts may be faced with the presently dormant problem of water rights litigation. Although competition for water supplies for public potable use has been kept out of the courts by specific legislation pertaining to the establishment of

rights for this purpose, no such legislation has yet been adopted to protect the farm irrigator in his use of surface waters. Partial control under specific legislation has been provided for ground waters in certain portions of the State, under which farmers may establish definite rights to the use of such waters. Studies are presently under way in New Jersey as well as in other humid states to revise and amplify existing water law to provide adequate control over all of the water resources of the state so that equitable allocation for all beneficial uses may be accomplished. It is to be expected that the engineer, in cooperation with public water supply officials, recreational interests, industrial management, health authorities and farm groups, will take an increasingly important place in the management and distribution of the water supplies which are so vital to the economy of the State of New Jersey.

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IRRIGATION AND DRAINAGE DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1615)

(Over)

PUMPING REQUIREMENTS FOR LEVEED AGRICULTURAL AREAS[a]

Closure by H. W. Adams

H. W. ADAMS,[1] M. ASCE.—The writer appreciates the views presented by Mr. Williams and the points he raises are worthy of consideration. Some of the points, however, appear to need some clarification.

Mr. Williams states that the economic analysis presented in the basic paper is not applicable to urban areas since even infrequent interior flooding "cannot be tolerated". It is believed that the question as to application of the method of analysis and that of the acceptance of some chance of flooding of urban areas by waters originating in the interior area are separate considerations.

The writer, in the basic paper, stated that the method of economic analysis was considered to be applicable to a variety of pumping installations, including those where sump storage was not available. In such instances only the peak runoff rate and the maximum pumping rate required to prevent ponding would be determined. It was shown in Table 6 of the basic paper that the maximum return on the investment would be realized with a station having an installed capacity of 0.25 inch per day although a station capacity up to about 0.4 inch per day would be economically justified. Accordingly, there is latitude for the application of engineering judgment in the selection of pumping capacity to be installed. It is still the view of the writer that the method of analysis is applicable for determination of the optimum pumping capacity to be installed at stations serving urban areas since this is basically an economic consideration. The question as to whether or not the capacity giving the maximum return on the investment should be recommended for installation because of the possibility of it being found inadequate under certain storm occurrences is a completely separate consideration.

The term "cannot be tolerated" used by Mr. Williams needs clarification. This term conveys the thought that the installed pumping capacity for stations serving urban areas must be sufficiently large to handle the runoff from any possible magnitude of storm without causing interior flooding. The writer knows of no storm water pumping station in any urban area where such a capacity has been installed and it is believed that a prudent consideration of the economics involved would preclude such an installation. In fact, even though the installed pumping capacity would be adequate to accommodate the runoff from a probable maximum storm there would still be the problem of conveying the runoff to the pumping station without flooding the protected area. Storm sewer systems serving urban areas normally are designed to handle

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a. Proc. Paper 1236, May, 1957, by H. W. Adams.

1. Asst. Chief, Planning Div., Civ. Works, Office of the Chief of Engrs., U. S. Dept. of the Army, Washington, D. C.

runoff from only a modest rainfall occurrence, the adopted design being selected from a careful consideration of the value of properties that may be subjected to possible flooding and the probable frequency of flooding of these properties. This is an economic determination. Some chance of flooding is usually accepted as a basis of design with the result that the sewer system surcharges when the design capacity is exceeded.

The pumping capacity installed at a number of existing storm water pumping stations serving protected urban areas where sump storage is not available has been predicated on the capacity of the sewer system to convey flows to the station. In most instances allowance also is made for the additional water reaching the station because of the surcharged sewer system and by overland flow. When the sewer system is adequate to convey the design flow without surcharge then the pumping capacity is based on the adopted design flow rates. One basis of design for storm water pumping stations serving protected urban areas is the selection of a rate of rainfall from a frequency analysis which occurs coincidentally with the occurrence of a river stage that requires activation of the pumping station.

IRRIGATION IN THE HUMID AREAS[a]

Discussion by Harry F. Blaney

HARRY F. BLANEY,[1] M. ASCE.—This interesting paper by Mr. Kimbrough calling attention to the water supply and other problems confronting irrigated agriculture in the Southeastern States, indicates the need for more research to improve irrigation practices in humid climates. One of the critical problems confronting the engineer responsible for planning irrigation systems in humid areas, arises from lack of sufficient sound basic data on which to base system design. In recent years, the United States Soil and Water Conservation Research Division has expanded its irrigation studies in cooperation with Agricultural Experimental Stations.

The author's statement that "In Mississippi supplemental irrigation was tried on several occasions and all proved to be failures", and the example given, "to show that irrigation was attempted approximately twenty years before the present era", would be misleading if applied to all the South.

A survey by Irrigation Engineer Williams(1) of the United States Department of Agriculture, in 1910, on need of irrigation in humid regions, states "It has been demonstrated that irrigation is profitable for such crops (fruits and vegetables) during the long continued dry spells in all parts of the humid region, including citrus fruits in Florida".

A study by Irrigation Engineer Stanley(2) in 1915 indicates that irrigated citrus fruits and truck crops were profitable in Florida and that irrigation was initiated in that State during the droughts of 1890 to 1893. The acreage of irrigated crops in Florida in 1915, estimated by Stanley, are shown in the following tabulation.

"Irrigated truck crops:	Acres
Surface irrigated	12,000
Subirrigated	2,500
Overhead spray	3,000
Irrigated Citrus Groves	8,000
Total	25,500"

The 1940 census showed a total irrigated acreage in the thirty-one humid Eastern states of approximately 600,000 acres. The 1950 census gives a figure of 1,516,000 acres and area irrigated has increased considerably since that date.

a. Proc. Paper 1352, September, 1957, by E. A. Kimbrough, Jr.
1. Irrigation Engr., Los Angeles, Calif., Soil and Water Conservation Research Div., A.R.S., U.S.D.A.

In 1950, the writer made a survey of irrigated areas in the Atlantic Coast States for the purpose of determining the consumptive water requirements during drought periods for different crops for use in the irrigation guides of the United States Soil Conservation Service. Since no measurements were available, consumptive use was estimated by the formula(3,4) $u = \frac{tp}{100} = kf =$ monthly consumptive use in inches was employed, where t = mean monthly temperature, p = per cent of daytime hours, k = monthly coefficient and f = monthly use factor. The following table shows the results for several areas.

Estimated Monthly Consumptive Use of Water by Irrigated Grass Pasture in Typical Southern Areas[1]

	Monthly Consumptive Use, Inches				
Month	Norfolk, Virginia:	Louisville, Kentucky :	Charleston, So. Carolina:	Mobile, Alabama:	Orlando, Florida:
March	2.4	2.3	3.0	3.0	3.4
April	3.0	3.0	3.5	3.5	3.7
May	4.3	4.3	4.5	4.6	4.7
June	4.8	5.2	5.0	5.0	4.9
July	5.1	5.6	5.7	5.5	5.5
Aug.	5.1	5.4	5.3	5.2	5.3
Sept.	4.0	4.1	4.2	4.2	4.3
Oct.	2.9	3.0	3.3	3.3	3.6

Figs. 1-a and 1-b of Mr. Kimbrough's paper showing drought periods should prove very useful in planning irrigation schedules. Table 1 could be improved by adding the acreage irrigated in the eight Southern States in 1954.

An obstacle in the path of future development of irrigation in the humid areas is the present status of the laws covering the use of water for irrigation. Most of the Eastern states operate under the legal riparian doctrine. According to this doctrine, every owner whose property touches a stream or lake has riparian rights.

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1. Williams, Milo B., Possibilities and Need of Supplemental Irrigation in Humid Regions. U. S. Dept. of Agri., 1911 Yearbook separate 570. 1912.

1. Computed by formula $u = kf$ = monthly consumptive use.(3,4)

2. Stanley, F. W., Irrigation in Florida, U.S.D.A. Bul. No. 462. Feb. 1917, Washington, D. C.

3. Blaney, Harry F., Consumptive Use of Water, Vol. 117, ASCE, Trans. 1952, pp. 949-973.

4. Blaney, Harry F. and Criddle, Wayne C., Determining Water Requirements in Irrigated Areas from Climatological and Irrigation Data. SCS-TP-96. U.S.D.A. Aug. 1950.

SALT BALANCE IN GROUND WATER RESERVOIR OPERATION[a]

Discussion by Robert O. Thomas

ROBERT O. THOMAS,[1] M. ASCE.—This paper has pinpointed a problem of immediate concern to engineers engaged in the planning of irrigation developments for lands where the use of ground water storage is essential to successful project operation. The writer has previously invited attention to the salt balance problem in connection with the planned utilization of ground water storage capacity.[2]

Generally, available waters, both surface and ground, contain mineral salts of calcium, magnesium, potassium, and sodium in varying amounts, present in the water in the form of carbonates, sulphates, and chlorides. These salts are derived from the passage of water over and through rock formations and result from solution and weathering action. After application of water on the land, that part which is not consumptively used or which does not drain off on the surface will percolate to lower depths. This percolate may or may not join the main underlying ground water body, dependent upon the geologic structure of the area. As a result of use, some salt compounds will be given up in promoting plant growth or in combining with soil elements. Conversely, percolating water will absorb other salt compounds in passing through the soil.

The chemical composition of ground water often is found to be quite different from that which would result from the free mixing of waters accumulating from various known sources. Movement of water into and through alluvial deposits creates conditions which are favorable for the occurrence of the phenomenon known as base exchange. Such action depends upon the presence of base exchange material in the soil structure penetrated by the downward percolating water.

Valley ground water reservoirs, particularly in the West, are usually bordered on two sides, and sometimes completely enclosed by mountain ranges from which both the water and the water-bearing materials of the valley fill are generally derived. A common form of progressive deterioration in quality of water occurs in valley reservoirs that are replenished in large part from water that has already been subjected to use and has absorbed soluble minerals.

Most irrigation developments have made use of such valley lands since they generally have a fairly constant slope to some central point, thus

a. Proc. Paper 1359, September, 1957, by David B. Willets and Charles A. McCullough.
1. Sacramento, Calif.
2. "Ground Water Development Symposium", Transactions, ASCE, Vol. 122, 1957, p. 441.

facilitating the distribution of water. The use of surface waters for irrigation has frequently been accompanied by a rise of the water table due to percolation of unused water. In some areas, also, the water table has risen under much of the lowland area due to lateral movement of ground water from higher irrigated lands.

Pumping from many of such reservoirs is a contributing factor to deterioration of water quality to the extent that the available storage capacity is emptied sufficiently to induce replenishment by contaminating waters. The problem of increase in salinity is not solved by limiting the application of irrigation water to the required consumptive use, even if practicable means could be developed to provide only such quantities of water as are required by growing crops. Adequate solutions will probably involve the provision of drainage facilities, operated so as to intercept as great a quantity of saline percolating waters as possible, prior to the time they reach the main body of ground water.

Such drains will interpose problems in effecting recharge of the ground water reservoir for cyclic storage; however, these problems are not insurmountable. It may be necessary to plan recharge and extraction operations so as to create artificial mounds, or troughs, separating various portions of the basin; or certain lands may be set aside for use solely as percolation areas to ensure the good quality of recharge waters. It is possible to construct drainage facilities in such manner that they may be closed during the periods that recharge operations are occurring. Other solutions will occur to the reader.

In many irrigated areas it will be found that, without adequate drainage, the lands will fail to support intensive irrigated agriculture due to salinization of the underlying soils. Such salinization may occur as a result of the accumulation of salts from applied irrigation water, or through the rise of the water table, or by the combination of both causes. Water used for irrigation, particularly in the West, may contain from 0.1 ton to five tons of mineral salts per acre-foot. With applied water generally amounting to from three to five acre-feet per acre, it is obvious that a large quantity of salt can be deposited on, and in, the soil in a short time.

The quantity of salt actually absorbed and used by the growing plant is extremely minor. Consequently, the entire soluble salt load is concentrated in the approximately one-third of the applied water remaining after the consumptive use demand has been satisfied. This concentration of the soil solution is much greater than that of the applied irrigation water. As subsequent water is applied to the land the concentrated solution is forced downward and, unless drainage is provided, eventually mixes with the water in storage. In many cases, rising water tables envelop this soil solution, causing swamp areas and high water table lands. Evaporation of ground water is increased and salt residues are left on the surface and in the upper soil layers.

Drainage of the accumulated concentrate will result in a major contribution to basin salt balance, assist in maintaining soil productivity, and to a large extent preclude deterioration of good quality ground water by dilution. The quantity of water to be drained or leached from the soil complex may be estimated by reference to Fig. 1. This diagram has been prepared from data published by the United States Department of Agriculture.[3] It has generally

3. "Diagnosis and Improvement of Saline and Alkali Soils", Agricultural Handbook No. 60, U.S.D.A., February, 1954.

been found that the electrical conductivity of the soil solution in the vicinity of the root zone should be no greater than 4,000 micromhos at 25° C. if salinization of the soil is to be prevented.

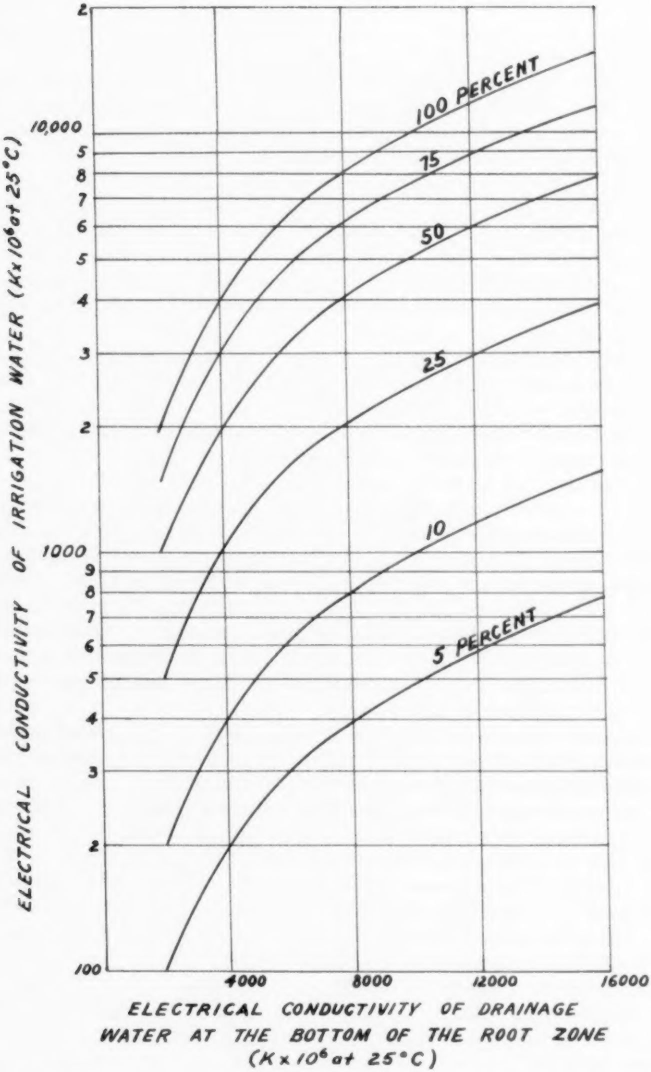
Soluble salts found in irrigation waters tend to create saline or alkali conditions in the soil when concentrations are permitted to increase abnormally. At such times the constituents of soil solutions may cause base exchange reactions that will result in altering the physical properties of the soil. Calcium and magnesium compounds generally tend to improve the permeability and friability of the soil. Sodium and potassium compounds, particularly the carbonates, chlorides, and sulphates, tend to inhibit plant growth and cause sealing, cracking, and gelatinization of the soil particles.

It is thus apparent that determined effort should be made to intercept and dispose of unconsumed irrigation water applied during the growing season. It is not necessary to waste all such water, however, since some drainage may be collected in strategically located sumps, diluted with other water of suitable quality, and again re-used in downstream areas. The economic limit to such re-use is, of course, the point where excessive quantities of diluting water are required.

The writer would classify saline connate waters trapped in sedimentary formations of marine origin, and magmatic and other juvenile waters rising along fault planes or through fractures, as natural sources of ofttimes serious problems of quality deterioration. Connate waters, particularly, have caused extensive damage in California's interior valleys.

While it is true, as stated by the authors, that substantial use can be made of the ground water storage capacity without unreasonable short-time deterioration of the stored water, it is also true that it is much easier to increase the salinity than to decrease it. In the third example given by the authors it would subsequently require 350,000 acre-feet of imported water at 400 p.p.m. and export of the same amount at 1,000 p.p.m., in addition to the normal import of 95,000 acre-feet and export of 38,000 acre-feet, to again reduce the average salinity to the value prevailing prior to the drought. The cost of quality deterioration is thus readily apparent. In the instant case, it would amount to about $50 per irrigated acre.

Provision of adequate drainage facilities would, it is believed, prove to be a more economical and satisfactory means of achieving a measure of salt balance than that gained by permitting deterioration of the quality of ground water to the extent used in the illustrative example presented in the paper.



LEACHING REQUIREMENT AS A PERCENTAGE OF APPLIED IRRIGATION WATER

Figure I

A GRAPHICAL SOLUTION FOR FLOW IN EARTH CHANNELS[a]

Discussions by R. G. Cox, Steponas Kilupaila and John F. Kennedy

R. G. COX,[1] A.M. ASCE.—Many graphical solutions of the Manning's equation for uniform flow have been developed to meet the special needs of hydraulic engineers. Mr. Carino's method is most suitable for trapezoidal channel problems involving single values of n, base width - depth ratio and side slope.

A graphical solution of the Manning's equation has been developed at the U. S. Army Engineer Waterways Experiment Station. The method, published in Hydraulic Design Criteria, permits rapid preliminary design work where different channel shapes and sizes, roughness values, and slopes are to be investigated. The Manning formula for open channel flow,

$$Q = \frac{1.486\, S^{1/2}\, R^{2/3}}{n}$$

was separated into a factor involving slope and friction

$$C_n = \frac{1.486\, S^{1/2}}{n}$$

and a geometric factor involving area and hydraulic radius

$$C_K = AR^{2/3}.$$

Thus

$$Q = C_n C_K$$

Two charts giving the value of C_n (illustrated by Fig. 1) for various slopes and roughness values were prepared. Companion charts giving the values of C_K for various shapes and base widths (illustrated by Fig. 2) were made. The channel size most suitable to right of way and hydraulic requirements can be determined in the following manner:

a. From Fig. 1 determine the value of C_n for the selected slope and roughness value.
b. Divide the design discharge by the value of C_n to get the required value of C_K.
c. Use the computed C_K value and Fig. 2 to select the most suitable channel geometry.

a. Proc. Paper 1360, September, 1957, by Isidoro D. Carino.
1. Chief, Analysis Section, Hydr. Analysis Branch, Waterways Experiment Station, Vicksburg, Miss.

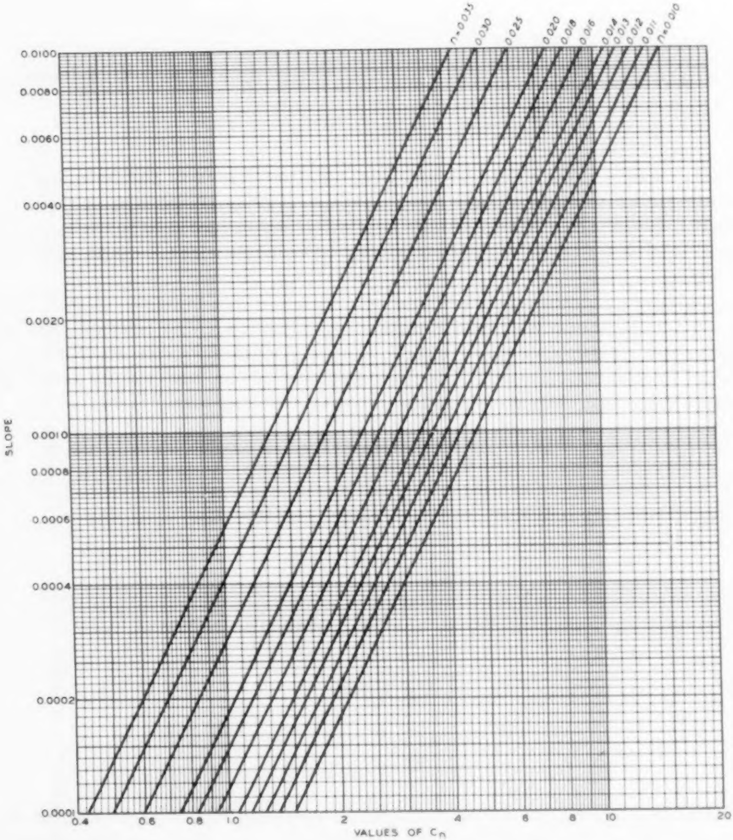


Figure 1. Values of C_n for various slopes and roughness coefficients

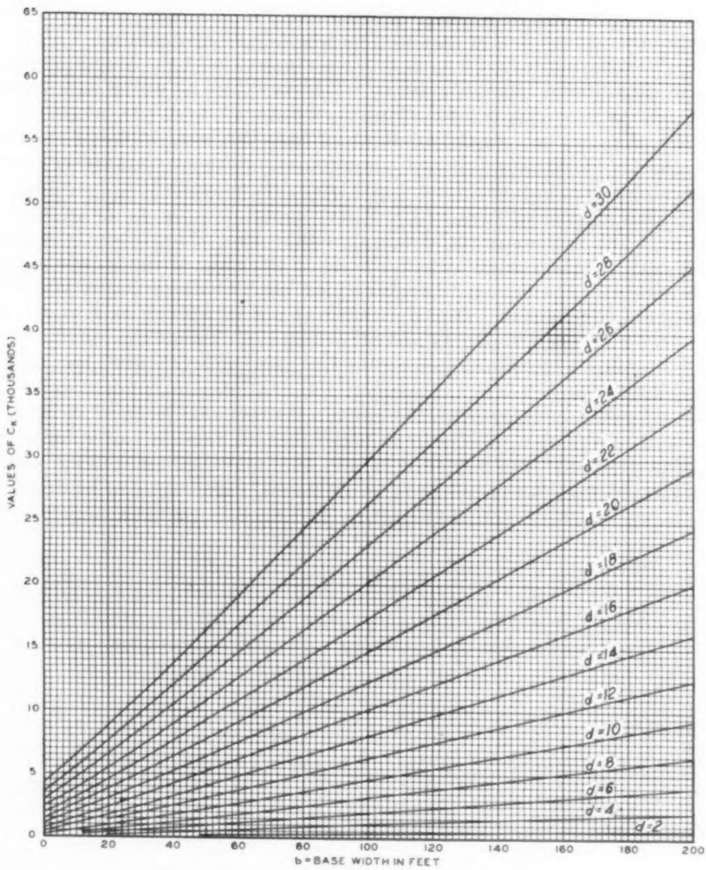


Figure 2. Values of C_k for various depths (d) and base widths (b). Trapezoidal channels with 1 to 1 side slope

STEPONAS KOLUPAILA[1] and JOHN F. KENNEDY.[2]—One of the most convenient tools for the solution of equations involving several parameters is the nomogram. When properly constructed and used, a nomogram can result in considerable saving of time by reducing the actual computation time required and also reducing the possibility of computational errors. It was interesting to learn from Mr. Carino's paper that the Bureau of Public Works in the Philippines is making use of this tool to simplify computation.

A very convenient form of nomogram for solution of the Manning equation applied to trapezoidal channels can be constructed as an array of four separate nomograms involving the following parameters: area (A), slope (S), discharge (Q), velocity (v), depth (h), width of base (b), side-slope of the channel (m), roughness coefficient (n), and combinations of these parameters into other parameters which will be defined later. It appears that a nomogram of this form was conceived first by the Russian engineer, A. L. Evfaritskii, in Central Asia.(1) A nomogram of this type (see Fig. 4) has been presented more recently by the senior writer.(2) As in the case of the author's presentation, a separate nomogram is required for each combination of Manning roughness and side-slope. This limitation is not as severe as it might first appear since both the roughness and side-slope of channels constructed in natural materials are usually determined by the material itself and often have corresponding values.

In the following development, the metric system will be used in keeping with the author's paper. It should be noted that all results can be applied to the English system if n is replaced by n/1.486, all other units being consistent.

Referring to Fig. 3, the following geometrical relations can be deduced:

$$A = (b + mh)\,h \tag{1}$$

$$P_w = b + Mh \tag{2}$$

where

$$M = 2\sqrt{1 + m^2} \tag{3}$$

and by definition,

$$R = \frac{A}{P_w} \tag{4}$$

At this point it is convenient to introduce the parameter

$$x = b/h \tag{5}$$

used by the author. This ratio, which was first used by Professor H. W. King,(3) is the most successful key for computation and standardization. Introducing x into Eqs. (1), (2) and (4), the following relations are obtained:

$$A = (x + m)\,h^2 = (x + m)\,\frac{1}{x^2}\,b^2 \tag{6}$$

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1. Prof. of Civ. Eng., Univ. of Notre Dame, Notre Dame, Ind.
2. Dept. of Civ. Eng., California Inst. of Technology, Pasadena, Calif.

$$P_w = (x + M)\,h \tag{7}$$

$$R = \frac{(x + m)}{(x + M)}\,h \tag{8}$$

Eq. (6) is the basis for construction of the third quadrant of the nomogram (Fig. 4). For constant values of h, A varies linearly with x, while for constant values of b, A varies hyperbolically with x.

The parameter used by the author

$$K = \frac{Q}{S^{1/2}} \tag{9}$$

often called the conveyance factor, is used as the abscissa of the first and fourth quadrants. For constant values of S, K varies linearly with Q and Eq. (9) is plotted as the straight lines of the first quadrant. When introduced into the Manning equation, Eq. (9) gives the formulation of the curves of the fourth quadrant. Thus

$$K = \frac{1}{n}\,\frac{(x + m)^{5/3}}{(x + M)^{2/3}}\,h^{8/3} \tag{10}$$

gives K as a function of x for constant values of h and

$$K = \frac{1}{n}\,\frac{(x + m)^{5/3}}{(x + M)^{2/3}}\,\frac{1}{x^{8/3}}\,b^{8/3} \tag{11}$$

gives K as a function of x for constant values of b.

The second quadrant is a plotting of the equation of continuity

$$Q = A\,V. \tag{12}$$

For constant values of v, Q varies linearly with A.

The construction of the nomogram is all quite straightforward with the exception of the fourth quadrant. The parameter Z defined as

$$Z = \frac{(x + M)^{1/4}}{(x + m)^{5/8}} \tag{13}$$

arises in connection with the formula for direct solution for the depth of flow in a trapezoidal channel

$$h = Z\,(n\,K)^{3/8} \tag{14}$$

and serves as an expedient in the construction of the curves for constant h. It will be noted that Z is a function of x and m only, and since the nomogram is constructed for only one value of m, it becomes a function of x only. Values of Z are presented in Table 1.

Substituting Eq. (13) in Eq. (10) yields

$$K = \frac{1}{n\,Z^{8/3}}\,h^{8/3} \tag{15}$$

Thus the curves of constant h are constructed by selecting a value of x, determining the corresponding value of Z and calculating K from Eq. (15) for the constant h and n. When the curves of constant h are plotted, the curves of

constant b are most easily constructed by making use of Eq. (5). There is a corresponding b for every value of x and h. The values of x for different values of h can be determined for any value of b and the loci of such points will give the curves of constant b.

Other information can also be incorporated into the nomogram. For example, the section requiring the minimum area, and thus the least excavation, can be shown to have a fixed value of x for each value of m. This section is often called the most efficient section. The most efficient channel will have a minimum P_w for a given A.

But from Eq. (2),

$$P_w = b + M h \tag{2}$$

and from Eq. (1),

$$A = (b + m h)h \tag{1}$$

Substituting Eq. (1) in Eq. (2)

$$P_w = \frac{A}{h} + (M - m) h \tag{16}$$

in which P_w is a function of only one variable, h, since A is being held constant. Differentiating,

$$\frac{d P_w}{dh} = - \frac{A}{h^2} + (M - m) = 0 \tag{17}$$

Substituting for A from Eq. (6),

$$x_1 = M - 2m. \tag{18}$$

For m = 1.5, x_1 = 0.606 which corresponds to a straight horizontal line across the third and fourth quadrants. This is shown as a dashed line on the nomogram. Values of x_1 for other values of m are given in Table 1.

The value of x = 2.5, which the author states is being used in the Philippines, together with a side-slope of 1.5 corresponds to what is commonly known as the American standard, for which

$$h = 0.5 \sqrt{A} \tag{19}$$

Substituting Eq. (19) into Eq. (6), the governing equation becomes

$$x = 4 - m. \tag{20}$$

Values of Z corresponding to the American standard are underlined in Table 1. Thus, for any m, the corresponding value of x will appear as a horizontal straight line across the third and fourth quadrants of the nomogram.

To illustrate the use of the nomogram, the examples used by the author will be repeated here:

Example 1

Given: Q = 3.5 cubic meters per second
x = 2.5

$S = 0.0005$

Required: h and v

Solution: Enter quadrant 1 with $Q = 3.5$ and intersect the line $S = 0.0005$. Proceed vertically downward to $x = 2.5$ where it is found that $b = 2.8$ meters, $h = 1.12$ meters. Proceed horizontally to the left, to the point $b = 2.8$, $h = 1.12$ where it is found that $A = 5.2$ sq. meters. Proceed vertically upward to the line $Q = 3.5$ where, by interpolation $v = 0.72$ meters per second.

Example 2

Given: $Q = 3.0$ cubic meters per second.
$x = 3$
$n = 0.025$
$m = 1.5$
$S = 0.0003$

Required: h

Solution: Enter quadrant 1 with $Q = 3.0$ and proceed to the left to intersect $S = 0.0003$. Proceed vertically downward to intersect $x = 3$ where it is found that $b = 3.24$ meters, $h = 1.08$ meters. Proceed horizontally to the left to the point $b = 3.24$, $h = 1.08$, where, from the abscissa, $A = 5.4$ square meters. Proceed vertically upward to the intersection of $Q = 3.0$ where $v = 0.57$ is interpolated between the velocity lines.

A wide variety of problems can be solved with this nomogram, all solutions consisting of essentially the same process outlined above. Each solution constitutes a rectangular path around the nomogram, the pertinent data being obtained in each quadrant. Additional data can be incorporated into the nomogram, such as formulas for minimum seepage loss and non-silting formulas. Nomograms can be constructed for any values of m and n, and on a scale to yield the desired accuracy, and can also be constructed for English units.

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TABLE 1

Values of Z (Eq. 13)

x	m = 0	m = 1	m = 1.5	m = 2	m = 2.5	m = 3
0	-	1.297	1.070	0.943	0.859	0.798
0.5	1.939	1.049	0.924	0.842	0.784	0.738
1	1.316	0.907	0.826	0.770	0.727	0.692
1.5	1.062	0.813	0.756	0.714	0.681	0.653
2	0.917	0.746	0.703	0.671	0.644	0.621
2.5	0.821	0.694	0.661	0.635	0.613	0.594
3	0.753	0.653	0.626	0.605	0.586	0.570
4	0.658	0.592	0.572	0.557	0.543	0.531
5	0.595	0.546	0.532	0.520	0.510	0.500
			Most Efficient Section			
x_1	2.000	0.828	0.606	0.472	0.385	0.325
Z	0.917	0.948	0.900	0.847	0.799	0.757

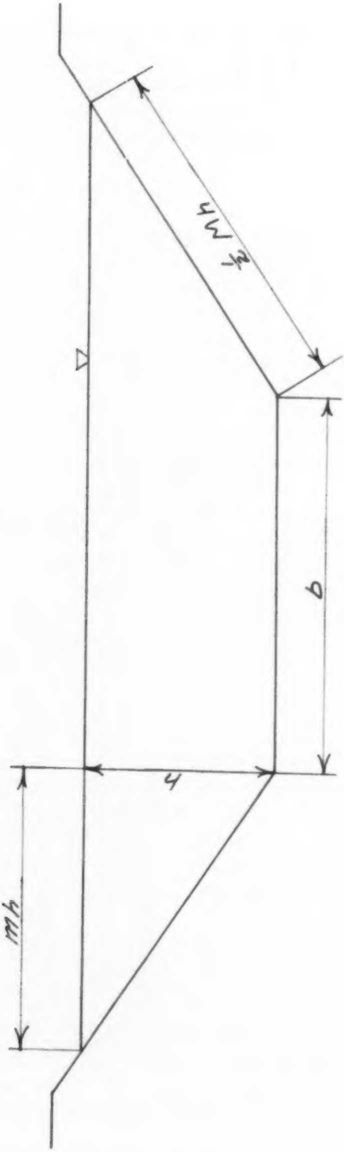

Fig 3

Nomogram for open channels in metric units, Manning formula

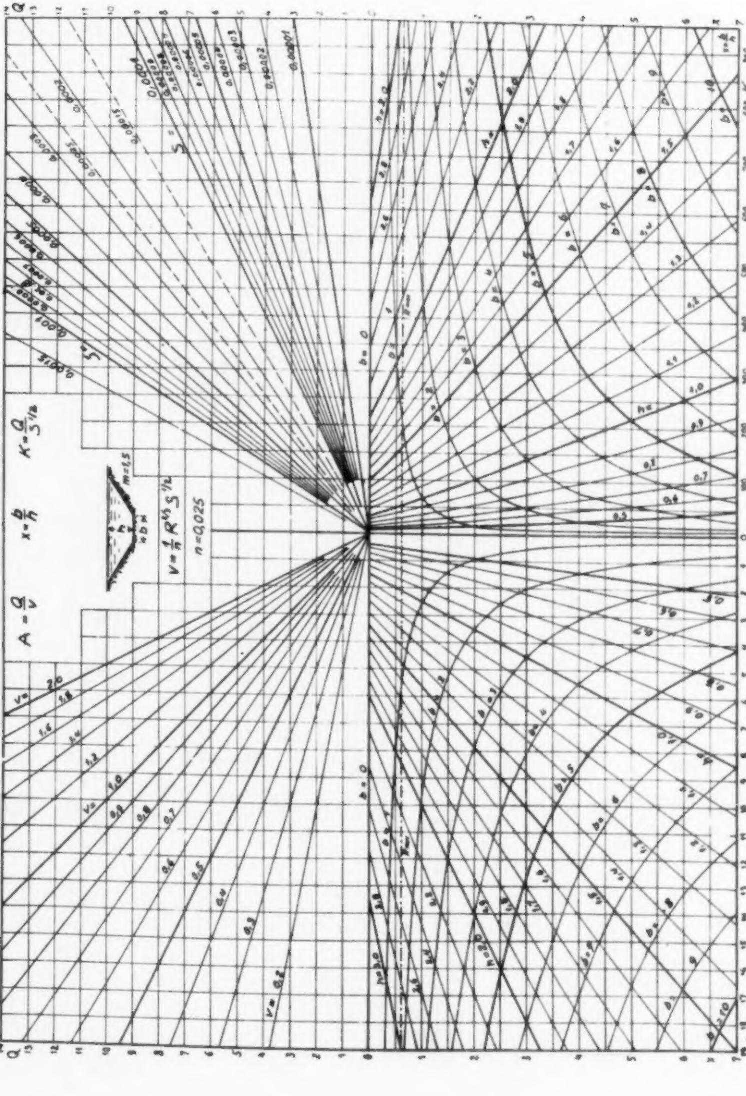

Fig 4

COMMON ERRORS IN MEASUREMENT OF IRRIGATION WATER[a]

Discussions by Ō. Starosolszky, Steponas Kolupaila and Armando Balloffet

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Ō. STAROSOLSZKY.[1]—The accuracy of flow measurement is closely related to the metering equipment used. Economic aspects indicate the necessity of investigations into the degree of accuracy and into possible errors—especially if water rates are based on similar observations.

Deficiencies dealt with by Mr. Thomas and encountered in practice are by no means accidental in nature and tend—in general—to result in regular errors. It should be noted, however, that accidental errors also occur which cannot be eliminated and have to be taken into account. It is deemed practicable, therefore, to establish limits for the resultant of accidental and regular (systematic) errors i.e., to specify the permissible tolerance. The measuring device cannot be accepted unless the deviation of the value observed during calibration from the theoretical one accepted as correct remains within the specified tolerance i.e., the accuracy limit expressed in per cent.

Individual errors encountered in practice can be traced back to errors in factors involved in the characteristic hydraulic relationship. The probable error in discharge can be computed from the error of individual factors in the discharge formula by the theorem of errors of Gauss. With the probable error E defined as the relative value,

$$E_Q\ \% = \sqrt{E_m^2 + E_b^2 + (aE_H)^2}$$

wherein E_Q, E_m, E_b, and E_H denote the probable errors of discharge, flow coefficient, characteristic length (e.g. the length of crest in case of weirs) and of stage observation respectively, the characteristic constant of the measuring device $\underline{a}$ (3/2 for rectangular weirs) is the exponent of the head $\underline{H}$ in the discharge relationship. An opportunity is thereby offered for summing up individual errors.

The relative probable error being inversely proportional to the head, poorest accuracies are obtained by any measuring device at low heads. The minimum head that can still be used for measuring should therefore be specified as a function of the permissible probable error, i.e., of tolerance, for each measuring device. The lowest head suitable for measurement—considering accidental errors only—depends on the graduation of the gage, the tolerance of discharge measurement, of the flow coefficient and on the constant $\underline{a}$. The minimum heads required for discharge measurement within accuracy limits

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a. Proc. Paper 1362, September, 1957, by Charles W. Thomas

1. Research Fngr., Hydr. Engr., Research Inst. for Water Resources, Budapest, Hungary.

(tolerance) of 3, 5, and 10 per cent respectively have been compiled in Table I for gage graduations of 8/10, 4/10 and 2/10 inches and for a 2 per cent error in the flow coefficient. As revealed by Table I no reliable results can be obtained in case of low heads. However, it is equally true that test-results are valid up to a certain head limit only. These considerations should be given due attention already during the design period.

High heads involve high head-losses. Consequently, for conditions prevailing on the Hungarian Plains, where gradients are mostly as low as 0.05 to 0.15 per mille, measuring devices involving small head-losses at suitably high heads had to be suggested.[2] Errors resulting from submergence are due partly to the inaccuracy introduced by the method of computation itself and partly to the unreliable determination of the limit of submergence. Influence of tailwater does not begin even in the case of sharp crested weirs when the tailwater reaches the level of the crest, since shooting flow can still develop, but at a well defined limit value of the tailwater-headwater ratio, i.e., at the limit submergence. The latter, however, depends on hydraulic conditions within the approach channel and on the degree of contraction as well as on the measuring device itself. A mathematical analysis of this kind of error is thus extremely difficult.

As pointed out by Mr. Thomas, head observations at sufficiently close intervals or continuous records (obtained by a recording gage) are essential to any reliable measurement of conveyed quantities, since even the most accurate device will fail to yield reliable results if the discharge observed is other than representative. The head varies during the period between successive observations as well. The error involved in the determination of flow quantities is therefore dependent upon the number of daily stage readings. In other words, a further error is introduced into the determination of flow quantities by computing for the period between observations with a discharge which is not representative for the flow actually passing during the period. The error in flow quantities is shown in Fig. 1 for a measuring device operating on the stage-difference principle (e.g. Venturi tube).

The depth of flow in the section under consideration is $\underline{d}$, the daily variation in stage is h_{daily} and thus the relative daily variation is

$$100 \frac{h_{daily}}{d}$$

and further, the number of daily readings is $\underline{n}$. The value of the increment-error has been assumed to equal the variation in stage between observations. Curves have been drawn for 2, 5 and 10 per cent errors in the discharge measurement E_Q. As is to be seen, readings taken once every day are in general unsatisfactory unless the stage fluctuation is very small. Efforts to maintain a fairly constant water level in the system are therefore of great significance. The necessary number of daily gage readings can also be seen in the Figure.

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2. Vizgazdálkodási Tudományos Kutató Intézet: Az öntözöviz mérése. Irrigation Water Measurement by Z. Karolyi and O. Starosolszky. Proceeding, Research Institute for Water Resources, No. 1. 1957—Budapest. For official use.

TABLE I

Relation between the tolerance in irrigation water measurement and the minimum head.

Name of measuring device		Probable error of head observation E_H and minimum head H_{min} / feet / in case E_m = 2 % discharge coefficient tolerance, for tolerances of discharge measurement								
	E_Q	3			5			1o		
		and for gage graduations of								
		8/1o	4/1o	2/1o	8/1o	4/1o	2/1o	8/1o	4/1o	2/1o
		i n c h e s								
Rectangular weir Venturi flume a = 3/2	E_H	1,48			3,o5			6,5		
	H_{min}	2,2	1,1	o,55	1,o8	o,54	o,27	o,5o	o,25	o,12
Thomson notch weir a = 5/2	E_H	o,9o			1,88			3,9		
	H_{min}	3,6	1,8	o,9	1,74	o,87	o,43	o,84	o,42	o,21
Linear weir and flume a = 1	E_H	2,23			4,58			9,8		
	H_{min}	1,46	o,83	o,42	o,72	o,36	o,18	o,34	o,17	o,o8
Venturi tube Submerged flow a = o,7	E_H	3,17			6,5o			13,9		
	H_{min}	1,o4	o,52	o,26	o,5o	o,25	o,12	o,24	o,12	o,o6

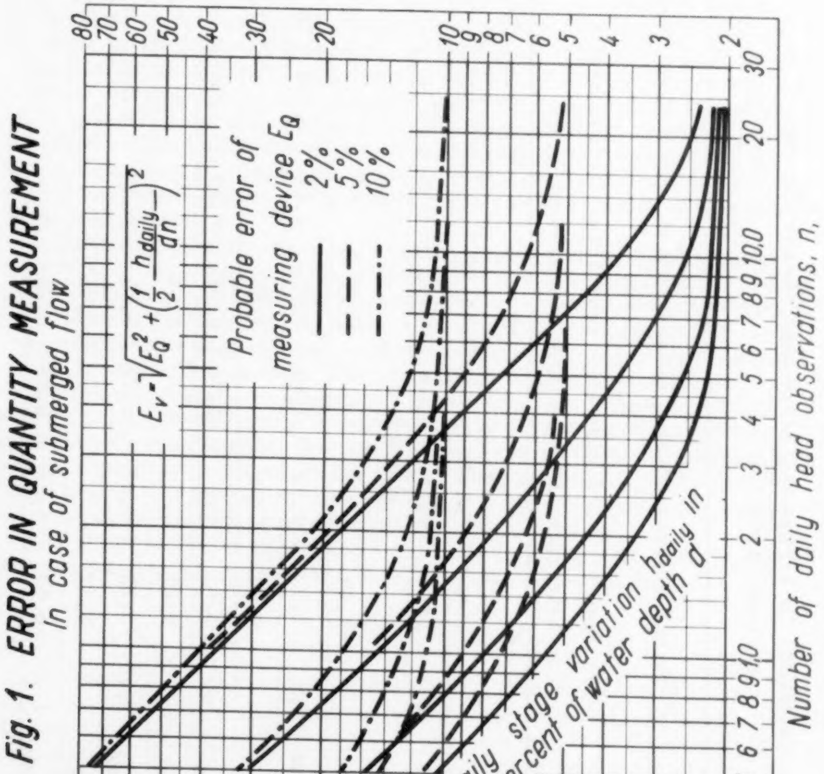

Fig. 1. ERROR IN QUANTITY MEASUREMENT
In case of submerged flow

In the light of the foregoing a tolerance of ± 10 to 20 per cent should be taken into consideration in present practice. The discrepancy due to accidental errors may attain the same magnitude,

STEPONAS KOLUPAILA.[1]—The valuable systematic work performed by the author invites some comments.

It is unfortunate that the list of references contains American articles only. A great research work on similar topics is to be credited to European engineers. Studies by B. Gentilini, R. Hailer, W. Dietrich, O. Dillman and others could be of particular interest. They found that a most dangerous error is caused by different velocity distribution before the weir.

Lack of information and bibliography explains that we sometimes duplicate the work already done by someone else.

The author applies the Rehbock formula in one of older forms (1912). During 17 years Professor Rehbock collected 280 new measurements and had improved his formula, which better corresponds to dimensional requirements. This new formula was published in 1929 and should replace the three older forms.

A deficiency in information does an injustice to the inventor of the trapezoidal weir. The name of the Italian engineer Cesare Cipolletti (1843-1908) is widely known around the world. A town in Argentina bears the name César Cipolletti in honor of the engineer who spent part of his life in service of government. His paper on the trapezoidal weir appeared in Italy in 1870, and then in English.(1)

His name was misspelled by our writers, like R. G. Hosea in Engineering Record, 26 (1892), Aug., p. 168, or A. D. Flinn and C. W. D. Dyer in Transactions, ASCE, 32 (1894), Paper No. 715, p. 9. The same mistake is admitted by the author.

The name of James Thomson (1822-1892), inventor of a triangular weir (V-notch in this paper) is often misspelled as Thompson.

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ARMANDO BALLOFFET.[1]—The fact that every measurement necessarily involves an error, leads in many cases to a fatalistic attitude in field operators, who tend to consider any error as unavoidable, and consequently all too often do not attempt any possible correction for the results of their field determinations. The author should be commended for bringing the subject again to the attention of the profession.

In the writer's mind, the error of measurement begins with an improper selection of the measuring device, on account of the operating conditions. The very extended practice of utilizing sharp-crested weirs for field measurements is perhaps a source of those unavoidable errors to which the author makes reference. Cippoletti, rectangular and V-notch weirs are accurate under laboratory conditions to precisions of about one per cent. However, when installed in an irrigation ditch or canal, the calibration conditions are irretrievably lost, unless extreme care is exerted to reproduce those laboratory conditions. This usually is impossible because of either cost of installation or maintenance. The photographs presented in the paper are most illustrative of how far from an accurate set-up field conditions can be.

The writer agrees with the author in that calibration of usual measuring devices should not generally be extrapolated beyond the range of observations. From the author's graphs it is seen that the most important errors come from failure to appraise the true approach velocity, either because of silting of the bottom upstream of a weir or to improper location of the gage.

Bearing in mind these disadvantages of the devices reported in the paper, the writer believes that the best field results can be expected from a device that:

a. Does not tend to cause deposition of sediments in the upstream channel.
b. Is able to produce the flow conditions required for the measurement by itself, i.e. is fairly independent of the upstream section.
c. Is accurate enough under adverse conditions.
d. Calibration can safely be extrapolated within the range of conditions usually encountered in practice.
e. Is inexpensive.

Weirs fail to meet all but perhaps the fifth of these conditions, in most common instances. They necessarily raise the level of the bottom of the

1. Tippetts-Abbett-McCarthy-Stratton, New York, N. Y., Lecturer in Civ. Eng., Columbia Univ., New York, N. Y.

canal and care must be taken to maintain the specified upstream depth. In addition, the approach velocity depends upon the section of the canal upstream of the weir, which may change due to said deposition, or to weeds, sliding of banks, etc. Their accuracy is directly affected by the usually off-laboratory conditions of irrigation operation. As a result of all these factors, unless rather expensive precautions are taken when installed or operated, their apparent simplicity is deceptive.

The same remarks apply to orifices.

On the other hand, the critical flow meters, usually called Venturi flumes, are apt to be installed in such a way that they do not cause an appreciable rise in the bottom elevation, which enables them to avoid most of the inconvenience derived from deposition of silt. They are usually easily prefabricated to contain in a single unit all the required approach conditions, and because they act as a control section, the discharge determines the upstream depth uniquely, in the same way as for a weir under laboratory conditions.

The most common critical flow meter in the United States is the Parshall flume, whose possible errors are discussed by the author. The Parshall flume is an excellent measuring device when used strictly within the range of discharges, heads and dimension ratii originally used when calibrated. Usually calibration cannot be extrapolated, mostly because the critical section is located in a convergent canal, and unless the angle of convergence and the remaining geometrical parameters are kept equal or proportional, the discharge coefficients must change substantially.

The writer made a review of several types of critical flow meters developed in Europe, in India and in Argentina,(1) which showed that in all cases care was taken to define precisely the control or critical section, in a rather long reach of rectangular and horizontal canal. De Marchi and Contessini's experiments,(2) as well as the writer's calibration of several models,(1) demonstrated that if this simple precaution is taken, even with the use of fairly abrupt transition curves at the entrance of the measuring section, meter equation fulfills the theoretical discharge relationship with an error of about 5% or less. This degree of precision is satisfactory in many field installations. The discharge coefficient to correct the theoretical equation was determined with an error of perhaps 1%. In the writer's experiments, several models showed a remarkable constancy of this empirical discharge coefficient with respect to discharge.

The theoretical equation is based upon the critical flow on the control section of the meter, and if this section is well defined, the Froude number upstream of it is related to the discharge coefficient by a numerical constant. This indicates that if Froude analogy is accomplished when extrapolating the calibration results to a different meter, the discharge coefficients should be equal. The results of field calibration of one of such meters with a discharge 39 times greater than the maximum laboratory discharge, and ratio of upstream depth to approach width 60% greater, showed no difference in discharge coefficients beyond the range of experimental errors. (About 1.5%)(1)

Due to the relatively small difference between the theoretical and the actual equations, when the critical section is well defined, Ruggiero and Giudici(3) have been able to design meters to fulfill a prescribed error law for a very wide range of discharges. The prescribed error law was such as to make the magnitude of the errors in discharge due to inaccurate gage readings, an inverse function of the annual frequency of discharges. In that way, the meters were able to give a weighted indication of the volumes measured throughout

the year. Although the critical sections for those meters were far from rectangular, laboratory tests showed a fairly good agreement between the predicted and the experimental rating curves.

Finally, since most of the accelerated and curvilinear flow is located within the narrowed section, the location of the measuring scale in these models needs not be as precise as for the Parshall flumes.

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CAN EVAPORATION LOSSES BE REDUCED?[a]

Discussion by Robert O. Thomas

ROBERT O. THOMAS,[1] M. ASCE.—The author has quite properly drawn attention to the value of underground storage of water as a method of decreasing evaporation losses. The writer has previously referred to such effects as one of the considerations in planned utilization of ground water storage.[2]

The principal ground water reservoirs of California are found in approximately 225 alluvium-filled valleys. The alluvium is of Quaternary age, composed mainly of continental alluvial fans and flood-plain deposits, with some inter-bedded lagunal and shallow marine sediments in the ground water basins bordering the coast and inland bays. In addition, there are a large number of small alluvium-filled valleys in the mountainous areas of the State which supply local water needs.

Ground water has been utilized in California since about 1870, when the first use was made of this resource in the Los Angeles area. By 1900, approximately 10,000 wells had been drilled in the South Coastal Basin area in southern California. Draft on the ground water resources of the State has increased at an accelerated rate since the turn of the century. For many years, the total use of ground water in California has exceeded that of any other State. By 1954, approximately 50 per cent of the water supplies which were applied to beneficial use in California were secured from ground water sources.

The estimated mean seasonal requirements for water in California, in 1954, were approximately 24,500,000 acre-feet, of which about 12,000,000 acre-feet was supplied from underground sources. During 1954, approximately 8.3 million acre-feet was withdrawn from the San Joaquin Valley and 1.2 million acre-feet from the Sacramento Valley. These two basins thus supplied about three-quarters of all water produced from the underground basins in California.

The planned utilization of underground storage must envision the lowering of the water table in the reservoir during a period of dry years and the subsequent recharge and storage of surplus flows during the ensuing wet period. Unless sufficient storage capacity is thus made available, a portion of the surface supply during the wet period must be wasted; on the other hand, too great a lowering will result in the inability to fully recharge the basin due to inadequacy of the available supply.

The capacity of the underground reservoir to provide cyclic storage for the

a. Proc. Paper 1499, January, 1958, by G. Earl Harbeck, Jr.

1. Sacramento, Calif.

2. "Ground Water Development Symposium", Transactions, ASCE, Vol. 122, 1957, p. 441.

long-time average annual supply is solely dependent upon the rate at which such surface supplies may be placed in storage in the underground basin. Present experimental projects in inducing such artificial recharge are being conducted in many localities, including experiments in the use of chemical additives and mechanical aids to maintain percolation rates, and in the use of recharge, or injection wells. Significant contributions to ground water are also made by percolation from surface conduits, from rivers and streams, and from deep percolation of water applied to lands for the irrigation of crops. Much recent interest has been aroused in the possibilities in recovery of large quantities of otherwise waste water through the reclamation and spreading of sewage.

In past years, many local irrigated areas in California have supplemented available surface supplies by pumping from underlying ground water basins. However, this practice has not resulted in augmentation of the local water supply, but rather in a change in the point of diversion. The total water supply available over a long-time period has not changed. Temporarily, in some areas, there has been an apparent increase in the available water supply, but this has resulted from mining ground water placed in underground storage over a period of many years. The water which has been extracted is not a true increase in the water supply as it is not capable of being replaced. At some time in the future, in these areas, there will be little or no water available to pump in order to balance deficient surface supplies during times of drought.

Suitable surface locations for conservation and control of water supplies are, as pointed out by the author, diminishing in number as development of such sites progresses. At the same time, agricultural, suburban, and recreational land uses are impinging on the remaining sites at a rapid rate. Increasing construction costs for major water control structures, and the inherent waste in dissipating costly supplies through evaporation, combine to enhance the natural advantages of utilizing available underground storage capacity for cyclic storage and regulation of the vast supplies necessary to maintain our national development.

Utilization of ground water reservoirs for the maximum benefit to the area involved is dependent upon adequate geologic and hydrologic data, from which the results to be expected as a consequence development can be estimated. Prerequisite to the full development of ground water storage is adequate technical information, including the basic data necessary for application of proved hydrologic principles. With adequate hydrologic data, reliable estimates can be formulated of the quantity of water that can be made available on a firm annual basis by coordinated or conjunctive operation of surface and subsurface storage or, alternatively, by operation of the ground water basin alone.

In the administration and operation of ground water storage capacity for conservation and use, an entirely new field of operational problems will necessitate major effort toward their successful solution. These problems may be divided into broad areas designated legal, management, hydrologic, and operational. The problems in each area will, of necessity, contain elements common to each of the other areas as the interrelationship is such that definite boundaries between the areas cannot be drawn.

Hydrologic and operational problems encountered in utilizing underground water storage capacity are almost inseparable as such, principally due to the

nature and extent of the usual ground water reservoir. In most cases, such reservoirs are far larger, both in area and in total storage capacity, than available surface reservoir sites. Natural annual recharge, however, is generally much less in proportion to the storage capacity, than the annual accretion to surface reservoirs. The problems encountered will relate to the conjunctive operation of surface and ground water storage, the economic aspects of such operation, cyclic variations in climate and water supply, maintenance of quality, operation of recharge facilities, etc.

Legal problems are discussed herein from the standpoint of the layman in procedures of law and regulation. It is evident that statutes facilitating the planned operation of ground water storage capacity must be formulated in consultation with the engineer in order that they may rest on sound hydrologic foundations. Problems created through changing the character of surface streams from influent to effluent, and vice versa, must be given consideration. The planned raising and lowering of water tables requires the backing of legal authority. Drainage problems, the eradication of phreatophytes, arrangement of pumping patterns, the use and disposal of recovered water supplies, land management, and the maintenance of quality of ground waters are but a few of the many considerations which must rest on a structure provided by legislation. It is imperative that legislation provide adequate protection to prior rights to the use of water, establish equitable bases for compensating owners of rights acquired in order to accomplish successful operation, and provide a sound financial basis for the planned conservation project.

In the management area a prime problem is the education and cooperation, not only of the water user as an individual, but of the public as a whole. Another management problem of importance is found in research activities looking toward the improvement of the natural and artificial recharge of important underground reservoirs. Other major functions of management such as provision of imported supplies from areas of surplus for replenishing depleted underground reservoirs, requires thorough engineering study and plans coupled with the necessary fiscal, legal, and administrative supports at all levels.

Comprehensive investigation of ground water storage possibilities, as advocated by the author, is a positive step in the full utilization of our water supply. It will provide the basis for the judicious discussion and eventual resolution of the complicated operational and legal problems briefly outlined above.

METHODS OF COMPUTING CONSUMPTIVE USE OF WATER[a]

Discussion by Robert O. Thomas

ROBERT O. THOMAS,[1] M. ASCE.—An interesting application of the Blaney-Criddle "f" factor was developed several years ago by the writer, as a result of investigations leading to a basis for estimating urban water usage in planning studies.

The investigation, except for San Diego in the southwest corner of the State, was confined to cities in the Central Valley of California. The populations concerned varied from 10,000 to 435,000 and, in an economic sense, covered a wide range of municipal developments. For each city, the total water input and output was determined, and the difference, termed "net retention", was computed. It is evident that the net retention is not synonymous with consumptive use, as some of the water retained, particularly from that portion applied to the ground, may return to the common water supply by deep percolation or unmeasured drainage.

The water input for the purpose of municipal use was the sum of both private and public water delivery in the area. In the instances where water was not metered to the consumer, but was measured at well fields or at points of delivery to the distribution system, the gross supply was reduced by 15 per cent as an adjustment for leakage from mains. Precipitation was taken from Weather Bureau records and assumed to be uniform over the area. Total water input to a municipality was then the sum of deliveries plus precipitation.

Water outflow consisted of the measured or estimated storm and sanitary drainage. For locations where storm drainage was not measured, runoff was estimated to be 54 per cent of the total quantity of precipitation, based upon studies at St. Louis by W. W. Horner. In combined systems, the total outflow was measured.

The net retention was then computed as the summation of public water supply plus private water supply plus precipitation minus sanitary sewage minus storm drainage. It was necessary, in some cases, to adjust measured quantities to account for differences in the areas served by both water supply and drainage facilities. This was accomplished on an area basis from maps of the communities concerned.

The resulting quantities of water input, outflow, and retention were then expressed in units of acre-feet per acre and tabulated by months for each city. Upon plotting the annual data in a form similar to Fig. 1, it was found that a high degree of correlation existed between the three elements. The relationship between retention and other pertinent factors was then developed, and it developed that the combination of temperature and retention formed the

a. Proc. Paper 1507, January, 1958, by Wayne D. Criddle.
1. Sacramento, Calif.

familiar loop relationship observed in studies of temperature and evaporation.

This, in turn, led to the attempt to correlate retention with the Blaney-Criddle factor "f", or mean monthly temperature times monthly per cent of annual daytime hours (t x p). For the months of April through October, an excellent relationship, shown in Fig. 2, was found to exist. Based upon the data collected, retention during the winter months was estimated to average 0.04 foot per acre per month. Annual retention, then, for conditions similar to those encountered in the study, can be estimated to amount to 0.20 foot per acre plus the varying quantities determined from Fig. 2 for the April-October period. The supply required to meet this requirement can then be estimated by reference to Fig. 1.

These diagrams have been found to be of value for master planning purposes. Municipal planning usually involves estimates of future population growth and population density. Such estimates can be expressed in terms of acreage devoted to urban purposes and estimates of future water supply requirements derived therefrom.

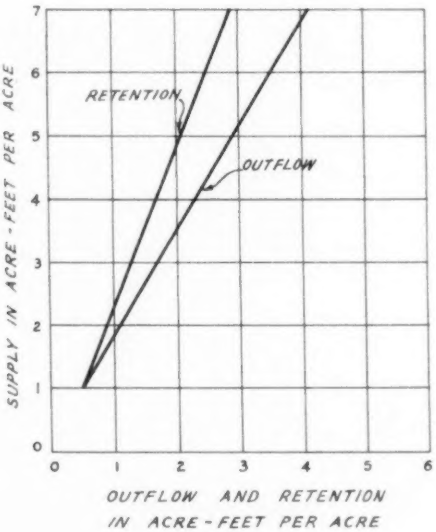
SUPPLY IN ACRE-FEET PER ACRE
RETENTION
OUTFLOW
OUTFLOW AND RETENTION
IN ACRE-FEET PER ACRE

FIG. 1

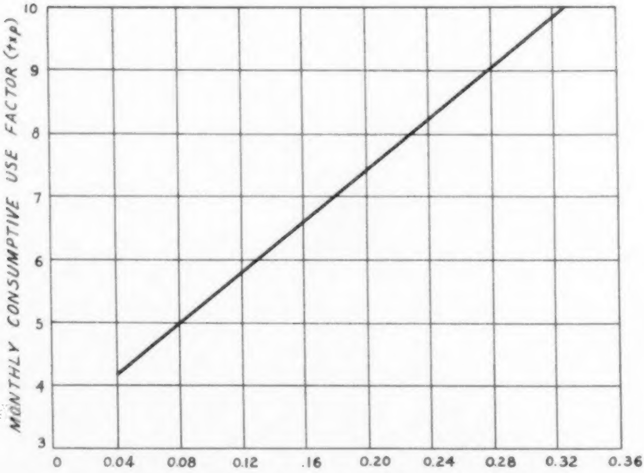
MONTHLY CONSUMPTIVE USE FACTOR (txp)
RETENTION IN ACRE-FEET PER ACRE

FIG. 2

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203(SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218(SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226(WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246(HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267(PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275(SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283(HY3)[c], 1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304(ST4), 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338(SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347(BD1), 1348(HY4)[c], 1349(SA4)[c] 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360(IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369(WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378(HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395(HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404(HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413(EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

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JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502(IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520(EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568(WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588(HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616(HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

c. Discussion of several papers, grouped by divisions.

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